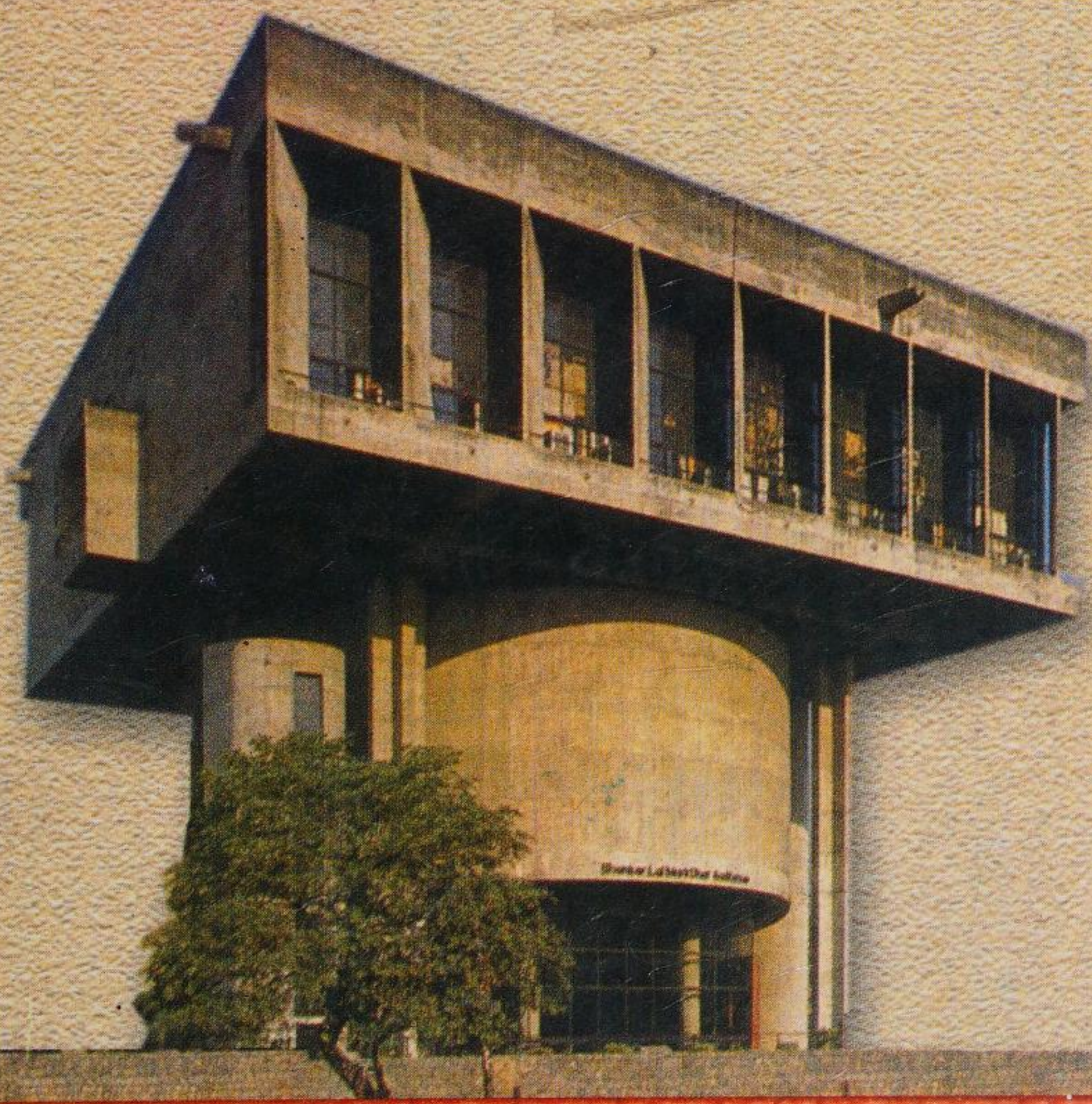
BUILDING CONSTRUCTION

Dr. B.C. PUNMIA



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A TEXTBOOK OF
BUILDING CONSTRUCTION

[AN ELEMENTARY AS WELL AS ADVANCED COURSE FOR ENGINEERING STUDENTS]

By

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Preface

Building Construction is a traditional science which deals with the modern methods of sound construction incorporating appropriate use of materials, sufficient strength and permanence, maximum utility, and good proportion and grace. The Building Design has traditionally been the responsibility of the Architect, though the Building Construction has been the responsibility of the Civil Engineer. However, the Structural Design of the buildings is the responsibility of a Civil Engineer. On small projects, a Civil Engineer may sometimes be entrusted with the architectural design work also, along with structural designs. The main considerations in the architectural design of buildings for all purposes are (*i*) climate and its effects, (*ii*) people and their requirements, (*iii*) materials for construction, and (*iv*) regulation and bye-laws of sanctioning authority. The aim of the present text book is to acquaint Civil Engineers, Architects, Builders, Contractors etc. with the basic principles as well as current design practices in the construction of buildings.

This book incorporates both an elementary as well as advanced course in Building Construction. The fist chapter is introductory, introducing various types of buildings and building components, as well as design loads. Chapters 2, 3 and 4 deal with the foundations–both shallow as well as deep. Though structural design of foundations has been avoided, the methods of proportioning of foundations appropriate to the loading and other site conditions, have been dealt with in greater details. Chapters 5, 6 and 7 deal with various types of masonry. Chapters 8, 9, and 10 are on various types of walls. In the past 20 years, the use of modern *Structural masonry* for multistoreyed building has been growing steadily following a period of partial eclipse by steel and concrete construction. Chapter 8 incorporates the structural design of tall load bearing walls of masonry. Chapters 11 and 12 deal with various types of floors while chapter 15 deals with various types of roofs and roof covering. Lintels and Arches are incorporated in chapter 13 while stairs are dealt with in chapter 14 ; in both these chapters, structural design has not been included. Chapters 16, 17 and 18 deal with carpentary and Joinery, Door and Windows and Shoring, Underpinning and Scaffolding. Chapters 19 and 20 describe the methods of Plastering, Pointing, Painting, Distempering and White Washing. Chapters 21 and 22 deal with Damp Proofing and Termite Proofing. Fire Protection has been discussed in Chapter 23 while Thermal Insulation has been dealt

with in chapter 24 with a number of illustrative solved examples. Chapters 25 and 26 deal with Concrete Construction. Chapter 27 deals with Ventilation and Air-conditioning methods. The Acoustic Design as well as methods of Sound Insulation are incorporated in Chapter 28. Lastly, the Management Methods through PERT and CPM networks have been dealt with in chapter 29.

The book uses both metric as well SI units. The book is based on current constructional practices prevalent in India, incorporating *latest* Indian Standard Recommendations. The basic construction features as well as design details have been profusely illustrated through neat sketches. It is hoped, the book will be useful to both the students as well as practising engineers.

I am thankful to Shri Kanhaiya Lal for nicely tracing all the illustrations. I am also thankful to the Publishers, for printing the book, with nice get-up, in such a short duration.

Jodhpur **B.C. PUNMIA**
1.8.84

Preface to the Second Edition

In the Second Edition of the book, the subject matter has been thoroughly revised and updated. A new chapter titled 'Water Supply and Drainage' has been added at the end of the book.

Jodhpur **B.C. PUNMIA**
2nd Jan. 1987.

Preface to the Fifth Edition

In the Fifth Edition of the book, the subject matter has been thoroughly revised, enlarged and updated. The entire book has been set up by DTP process. Further suggestions will be greatly appreciated.

Jodhpur **B.C. PUNMA**
Mahaveer Jayanti **ASHOK KUMAR JAIN**
5th April 93 **ARUN KUMAR JAIN**

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1

Introduction

1.1. GENERAL

Man requires different types of buildings for his activities: houses, bungalows and flats for his living ; hospitals and health centres for his health ; schools, colleges and universities for his education; banks, shops, offices, buildings and factories for doing work ; railway buildings, bus stations and air terminals for transportation ; clubs, theatres and cinema houses for recreation, and temples, mosques, churches, dharmshalas etc. for worship. Each type of the above buildings has its own requirements. The above building activities are an important indicator of the country's social progress.

Houses, bungalows, flats, huts etc. provide shelter to man. The first hut with bamboos and leaves can be taken as the first civil engineering construction carried out to satisfy the needs for a shelter. Before that, caves were his early abode. The history of development of housing facilities reveals that man has been moulding his environment throughout the ages, for more comfortable living. India still has many old cave temples with halls and rooms having beautiful carvings. Egyptians constructed huge pyramids. The Greeks developed a style of proportions of building elements ; these proportions are known as the *Orders of Architecture.* Romans developed arches for vaults and domes. They used pozzolana sand, mortar, plaster and concrete. During the Gothic period of architecture(1100-1500 A.D.) churches with pointed arches and the ribs supporting masonry vaults were constructed. The arched ribs were supported by stone pillars strengthened by buttresses. These structures led to the idea of framed structures.

The period from 1750 A.D. onwards is known as the period of Modern Architecture. Due to economic pressure after the war, and due to industrial development , many new methods and materials of construction were developed. The use of reinforced concrete construction triggered the rapid development of modern architecture.

Functional structural components such as columns, chajjas, canopies, R.C.C. slabs became increasingly popular because of the increased speed in construction. Use of plywood, glass, decoratives etc. helped the designers to make the new structures look more elegant.

The building design has traditionally been the responsibility of the architect, though the building construction has been the responsibility of the civil engineer. Also, the structural designs of the building are the responsibility of a civil engineer. On small projects, a civil engineer may sometimes be entrusted with the architectural design work, along with structural designs. The main considerations in architectural design of buildings for all purposes are as follows :

(1) Climate and its effect,
(2) People and their requirements,
(3) Materials for construction and method of construction, and (4) Regulations and bye-laws of sanctioning authority.

1.2. TYPES OF BUILDINGS

National Building Code of India (SP : 7–1970) *defines* the building as 'any structure for whatsoever purpose and of whatsoever materials constructed and every part thereof whether used as human habitation or not and includes foundations, plinth, walls, floors, roofs, chimneys, plumbing and building services, fixed platforms, verandah, balcony cornice or projection, part of a building or any thing affixed thereto or any wall enclosing or intended to enclose any land or space and signs and outdoor display structures'. Tents, shamianas and tarpaulin shelters are not considered as building.

According to the National Building Code of India (1970), Buildings are classified, based on *occupancy,* as follows:

Group A : Residential buildings.
Group B : Educational buildings.
Group C : Institutional buildings.
Group D : Assembly buildings.
Group E : Business buildings.
Group F : Mercantile buildings.
Group G : Industrial buildings.
Group H : Storage buildings.
Group J : Hazardous buildings.

1. Group *A* : Residential Buildings

These are those buildings in which sleeping accommodation is provided for normal residential purposes, with or without cooking or dining or both facilities, except any building classified under category *C*. Buildings of group *A* are further sub-divided as follows :

(i) Sub-division A-1 : Lodging or Rooming Houses

These include any building or group of buildings under the same management, in which separate sleeping accommodation for a total of not more than 15 persons, on either transient or permanent basis with or without dining facilities, but without cooking facilities for individuals, is provided.

A lodging or rooming house is classified as a dwelling in sub-division *A-2* if no room in any of its private dwelling units is rented to more than three persons.

(ii) Sub-division A-2 : One or two Family Private Dwellings

These include any private dwelling which is occupied by members of a single family and has a total sleeping accommodation for not more than 20 persons.

If rooms in a private dwelling are rented to outsiders, these should be for accommodating not more than 3 persons.

If sleeping accommodation for more than 20 persons is provided in any one residential building, it should be classified as a building sub-division *A-3* or *A-4* as the case may be.

(iii) Sub-division A-3 : Dormitories

These include any building in which group sleeping accommodation is provided, with or without dining facilities, for persons who are not members of the same family, in any one room or a series of closely associated rooms under joint occupancy and single management, for example, school and college dormitories, students and other hostels and military barracks.

(iv) Sub-division A-4 : Apartment Houses (Flats)

These include any building or structure in which living quarters are provided for three or more families living independently of each other and with independent cooking facilities, for example, apartment houses, mansions and chawls..

(v) Sub-division A-5 : Hotels

These include any building or group of buildings under single management in which sleeping accommodation, with or without dining facilities, is provided for hire to more than 15 persons who are primarily transient, for example hotels, inns, clubs and motels.

2. Group *B* : Educational Buildings

These include any building used for school, college, or day-care purposes for more than 8 hours per week involving assembly for instruction, education or recreation and which is not covered by Group *D*.

3. Group *C* : Institutional Buildings

These include any building or part thereof, which is used for purposes such a medical or other treatment or care of persons suffering

from physical or mental illness, disease or infirmity ; care of infants, conval escents or aged persons and for penal or correctional detension in which the liberty of inmates is restricted. Institutional buildings ordinarily provide sleeping accommodation for the occupants.

Buildings under group *C* are further sub-divided as follows:

(*i*) ***Sub-division C-1 : Hospitals and Sanitaria***

This sub-division includes any building or group of buildings under single management, which is used for housing persons suffering from physical limitations because of health or age, for example, hospitals, infirmaries, sanitaria and clinics.

(*ii*) ***Sub-division C-2 : Custodial Institutions***

This sub-division includes any building or group of buildings under single management, which is used for the custody and care of persons such as children, convalescents and the aged, for example, homes for the aged and infirm, convalescent homes and orphanages.

(*iii*) ***Sub-division C-3 : Penal Institutions***

This sub-division includes any building or a group of buildings under single management, which is used for housing persons under restraint, or who are detained for penal or corrective purposes, in which the liberty of the inmates is restricted, for examples, jails, prisons, mental hospitals, mental sanitaria and reformatories.

4. Group *D* : Assembly Buildings

These include any building or part of a building, where group of people congregate or gather for amusement, recreation, social, religious, patriotic, civil, travel and similar purpose, for example, theatres, motion picture houses, assembly halls, auditoria, exhibition halls, museums, skating rinks, gymnasiums, restaurants, places of worship, dance halls, club rooms, passenger stations and terminals of air, surface and marine public transportation service, recreation piers and stadia.

Buildings under group *D* are further sub-divided as follows:

(*i*) ***Sub-division D–1***

This sub-division includes any building primarily meant for theatrical or operatic performances and exhibitions and which has a raised stage, proscenium curtain, fixed or portable scenery or scenery loft, lights, motion picture booth, mechanical appliances or other theatrical accessories and equipment and which is provided with fixed seats over 1000 persons.

(*ii*) ***Sub division D–2***

This sub-division includes any building primarily meant for use as described for sub-division *D–1* but with fixed seats for less than 1000 persons.

(iii) Sub-division D–3

This sub-division includes any building, its lobbies, rooms and other spaces connected thereto, primarily intended for assembly of people, but which has no theatrical stage or theatrical and/or cinematographic accessories and has accommodation for more than 300 persons, for example, dance halls, night clubs, halls for incidential picture shows, dramatic, theatrical or educational presentation; lectures or other similar purposes, having no theatrical stage except a raised platform and used without permanent seating arrangement ; art galleries; museums; lecture halls; libraries ; passenger terminals and buildings used for educational purposes for less than 8 hours per week.

(iv) *Sub-division D–4*

This sub-division includes any building primarily intended for use as described in sub-division *D–3* but with accommodation for less than 300 persons.

(v) *Sub-division D–5*

This sub-division includes any building meant for outdoor assembly of people not covered by sub-division *D–1* to *D–4*, for example, grand stands, stadia, amusement park structures, reviewing stands and circus tents.

5. Group *E* : Business Buildings

These include any building or part of a building, which is used for the transaction of business(other than that covered by building in Group *F*); for the keeping of accounts and records and similar purposes; doctors' and dentists' (unless these are covered by the provisions of Group *C*); service facilities, such as new stands, lunch counters serving less than 100 persons, barber shops and beauty parlours.

City halls, town halls, court houses and libraries should be classified in this group in so far as the principal function of these is transaction of public business and the keeping of books and records.

Minor office occupancy incidental to operation is another type of occupancy should be classified under the relevant group for main occupancy.

6. Group *F* : Mercantile buildings

These include any building or part of a building, which is used as shops, stores, markets, for display and sale of merchandise, either wholesale or retail.

Office, storage and service facilities incidental to the sale of merchandise and located in the same building should be included under this group.

Minor merchandising operations in buildings primarily meant for other uses should be covered by group under which the predominant occupancy is classified.

7. Group *G* : Industrial Buildings

These include any building or part of a building, or structure in which products or materials of all kinds and properties are fabricated, assembled or processed, for example, assembly plants, laboratories, dry cleaning plants, power plants, pumping stations, smoke houses, gas plants, refineries, dairies and saw mills.

8. Group *H* : Storage Buildings

These include any building or part of a building, used primarily for the storage or sheltering (including servicing, processing or repairs incidental to storage) of goods, wares or merchandise (except those that involve highly combustible or explosive products or materials), vehicles or animals, for example, warehouses, cold storages, freight depots, transit sheds, store houses, truck and marine terminals garages, hangers (other than aircraft repair hangars), grain elevators, barns and stables.

9. Group *J* : Hazardous Buildings

These include any building or part of a building which is used for the storage, handling, manufacture or processing of highly combustible or explosive materials or products which are liable to burn with extreme rapidity and/or which produce poisonous fumes or explosions; for storage, handling, manufacturing or processing which involve highly corrosive, toxic or noxious alkalies, acids or other liquids or chemicals producing flame, fumes and explosive, poisonous, irritant or corrosive gases; and for the storage, handling or processing of any material producing explosive mixtures of dust or which result in division of matter into fine particles subject to spontaneous ignition. Examples of buildings in this class are those buildings which are used for :

(*a*) Storage under pressure of more than 0.1 N/mm^2 and in quantities exceeding 70 m^3 of acetylene, hydrogen, illuminating and natural gases, ammonia, chlorine, phosgene, sulphur dioxide, carbon dioxide, methyl oxide and all gases subject to explosion, fume or toxic hazard;

(*b*) Storage and handling of hazardous and highly flammable liquids;

(*c*) Storage and handling of hazardous and highly flammable or explosive materials other than liquids; and

(*d*) Manufacture of artificial flowers, synthetic leather, ammunition, explosives and fireworks.

1.3. COMPONENTS OF A BUILDING

A building has two basic parts:

(*i*) Sub-structure or foundations, and

(*ii*) Super-structure.

Sub-structure or Foundation is the lower portion of the building, usually located below the ground level, which transmits the loads of the super-structure to the supporting soil. *A foundation is therefore that part of the structure which is in direct contact with the ground to which the loads are transmitted.*

Super-structure is that part of the structure which is above ground level, and which serves the purpose of its intended use. A part of the super-structure, located between the ground level and the floor level is known as *plinth.*

Plinth is therefore defined as the portion of the structure between the surface of the surrounding ground and surface of the floor, immediately above the ground. The level of the floor is usually known as the *plinth level.* The built-up covered area measured at the floor level is known as *plinth area.*

A *building* has the following components :

1. Foundations.
2. Masonry units : walls and columns.
3. Floor structures.
4. Roof structures.
5. Doors, windows and other openings.
6. Vertical transportation structures, such as stairs, lifts, ramps etc.
7. Building finishes.

1. Foundations

The basic function of a foundation is to transmit the dead loads, live loads and other loads to the subsoil on which it rests in such a way that (*a*) settlements are within permissible limits, without causing cracks in the super-structure and (*b*) soil does not fail in shear. Since it remains below the ground level, the signs of failure of foundations are not noticeable till it has already afffected the building. It should therefore be designed very carefully. Various types of foundations and their design principles have been discussed in chapters 2, 3 and 4.

2. Masonry units : walls, columns etc.

Masonry may be defined as the construction of building units bonded together with mortar. These building units, commonly known as *masonry units* may be stones, bricks or precast blocks. Masonry is used for the construction of foundation walls, columns and other

similar structural components. The construction with stone units, bonded with mortar is known as *stone masonry,* while the construction with brick units, bonded with mortar is known as *brick masonry. A composite masonry* may use different types of building units for the construction.

Walls are the most essential components of a building. The primary function of the wall is to enclose or divide space of the building to make it more functional and useful. Walls provide privacy, afford security and give protection against heat, cold, sun and rain. Walls may be either load bearing or non-load bearing. Load bearing walls are those which are designed to carry the super-imposed loads (transferred through roofs), in addition to their own(self) weight. Non-load bearing walls carry their own load only. They generally serve a *divide walls or partition walls.* Wall may be of several types, such as cavity walls, party walls, partition walls, dwarf walls, retaining walls. These have been discussed in chapter 8, 9 and 10.

A *column* is an isolated vertical load bearing member, the width of which is neither less than its thickness nor more than four times its thickness. A *pier* is a member similar to a column except that it is bonded into load bearing wall at the sides to form integral part and extends to the full height of the wall. A pier is used to increase the stiffness of the wall to carry additional load or to carry vertical concentrated load.

3. Floor structures

Floors are the horizontal elements which divide the building into different levels for the purpose of creating more accommodation within a restricted space one above the other and provide support for the occupants, furniture and equipment of a building.

The floor of a building immediately above the ground is known as *ground flocr.* All other floors which are above the ground floor are known as the *upper floors.* The floors of the first storey is known as the *first floor* and that of the second storey is known as the second floor etc, etc. In case, part of the building is constructed below the ground level, or the building has the basement, the floor is known as *basement floor.*

Every floor has two components : (*i*) the *sub-floor,* which is a structural component to impart strength and stability to support the super-imposed loads and (*ii*) *floor covering or flooring* consisting of suitable floor finish.

Floor area is the usable covered area of a building at any floor level.

Floor area ratio (F.A.R.) is defined as the quotient obtained

by dividing the total covered area (plinth area) on all floors and 100 by the area of the plot:

$$\text{Thus, F.A.R.} = \frac{\text{Total area covered of all floors} \times 100}{\text{Plot area}}$$

4. Roof structures

A roof is the upper most part of a building. It is a covering provided on the top of the building with a view to keep out rain, snow, sun and wind and to protect the building from their adverse effects. Just as a floor, a roof consists of two components : (*i*) The roof decking and (*ii*) the roof covering. *Roof decking* is a structural component which supports the roof covering. Roof decking may be either flat or sloping, and may be in the form of flat slab, dome, truss, portal or shell. The *roof covering or roofing* is provided on the roof deck to safeguard the building against weather effects. These may be in the form of tiles, thatch covering, slates, flagstone covering, and corrugated sheets of galvanised iron or asbestos cement.

5. Doors, Windows and other Openings

A *door* is a movable barrier provided in the opening of a wall, to provide access to various spaces of a building. A door is a frame work of wood, steel etc. secured in the wall opening for the purpose of providing access to the users of the building. Similarly, a window may be defined as an opening made in the wall for the purpose of providing day light, vision and ventilation. Windows are also made of frame work of wood, steel, aluminium etc., provided with shutters.

Since doors and windows are provided in the openings in the walls, a discontinuity is formed in the wall, in the vertical direction. *Lintels* are therefore essential. A *lintel* is a horizontal structural member provided over the doors, windows or other openings, to span the gap, so as to support the super-imposed load carried by the wall above the opening. Lintels may be made of timber, stone, steel or reinforced cement concrete (R.C.C.).

Sometimes, an *arch* may be provided to span the opening, in the place of a lintel. An arch is a structure consisting of a number of small wedge-shaped units and jointed together with mortar, which is constructed to bridge across any opening in the wall. The arch may also be constructed in R.C.C.

6. Vertical Transportation Structures

These consists of stairs, ramps, ladders, lifts and escalators etc. to afford access between various floors. Out of these, stairs

are the most common. A *stair* may be defined as series of steps suitably arranged for the purpose of connecting different floors of a building. Alternatively, a *stair* may be defined as an arrangement of treads, risers, stringers, newel posts, hand rails and balustrades so designed and constructed as to provide an easy, safe and quick access to the users of different floors. Stairs may be constructed of different materials such as timber, stone, reinforced concrete or steel.

7. Building Finishes

Building finishes are used to give protective covering to various building components, and at the same time, they provide decorative effects. Building finishes consists of the following items:

(*i*) Plastering
(*ii*) Pointing
(*iii*) Painting
(*iv*) Varnishing and polishing
(*v*) White washing
(*vi*) Distempering
(*vii*) Colour washing or colouring.

Plastering consists of providing a thin covering of plastic materials such as cement mortar, lime mortar etc. on walls, columns and other surfaces. *Pointing* is the process of finishing of mortar joints in brick or stone masonry. *Painting, varnishing and polishing* is normally done on doors, windows and other timber and steel components. *White washing, distempering* and *colour washing* etc. are done on plastered surfaces, to safegaurd them against weathering effects and to improve the appearance.

1.4. DESIGN LOADS

The basic requirement of any structural component of a building is that it should be strong enough to carry or support all possible types of loads to which it is likely to be subjected. Loads coming on a structure may be of three types :

1. Dead loads,
2. Live loads, and
3. Wind Loads.

1. DEAD LOADS

The dead load in a building shall comprise of the weight of all walls, partitions, floors and roofs and shall include the weights of all other permanent construction in the building.

The unit weights of some common materials, as per IS : 1911–1967 are given in Table 1.1.

TABLE 1.1 UNIT WEIGHTS OF SOME COMMON MATERIALS
(IS : 1911–1967)

Material	*Weight* (kg/m^3)
1. Bituminous substances	
Anthracite coal	1500
Peat (dry)	560 to 640
Charcoal (light)	300
Coke	1000
Crude oill	880
Pitch	1010
Coal tar	1000
2. Excavated materials	
Clay (dry, compact)	1440
Clay (damp, compact)	1760
Earth (dry)	1410 to 1840
Earth (moist)	1600 to 2000
Sand (dry)	1540 to 1600
Sand	1760 to 2000
3. Liquids	
Alcohol	780
Gasoline	670
Ice	910
Nitric acid 91%	1510
Sulphuric acid 87%	1790
Vegetable oil	930
Water (fresh)	1000

Material	*Weight* (kg/m^3)
4. Building materials	
Bricks	1600 to 1920
Cement (ordinary)	1440
Chalk	2240
Glass	2400 to 2720
Lime stone	2400 to 2640
Sand stone	2240 to 2400
Steel	7850
Timber	650 to 720
5. Structural items, ceilings, finishes etc.	
Asbestos cement sheets	12 to 15.6
Brick masonry	1920
Brick wall, 100 mm thick	192
Brick wall, 200 mm thick	384
Brick wall, 300 mm thick	576
Cement plaster, 25 mm thick	52
Concrete, plain	2300
Concrete, reinforced	2400
Dry rubble masonry	2080
Galvanised iron sheet,	
0.50 mm thick	5 (kg/m^2)
1.63 mm thick	13 (kg/m^2)
Mangalore tiles with battens	63 (kg/m^2)

[Note : 1 $kg/m^3 \approx 10$ N/m^3

2. LIVE LOADS

Live loads, also called as super-imposed loads, consist of moving or variable loads, due to people or occupants, their furniture, temporary stores, machinery etc.

Live loads on Floors

Live loads on floors shall comprise of all loads other than dead loads. The minimum live loads on different floors for different uses shall be as given in Table 1.2. The loads specified in Table 1.2 are uniformly distributed static loads in kg/m^2 (kN/m^2) on the plane area and provide for normal effects of impact and acceleration, but do not take into consideration special concentrated loads, snow loads and other loads.

In the case of multistoreyed buildings, the following reductions in live loads may be made in designing column, walls, piers and foundations :

No. of floors carried by member under consideration	*Percent reduction of total live load on all floors above the member under consideration*
1	0
2	10
3	20
4	30
5 or more	40

No reduction shall be made in the case of warehouses, garages, and other buildings used for storage purposes, and for factories and workshops, designed for $500\ kg/m^2$ ($5\ kN/m^2$). However, for buildings, such as factories and workshops, designed for a live load of more than $500\ kg/m^2$ ($5\ kN/m^2$), the reduction given above may be made, provided that the loading assumed for any column, wall etc. is not less than that which would have been if all floors had been designed for $500\ kg/m (5\ kN/m^2)$ with no reduction.

TABLE 1.2. LIVE LOADS ON FLOORS (IS: 875-1964)

Loading Class No. (1)	*Types of floors* (2)	*Min. Live Load per m^2 of floor area* (3)	*Alternative minimum live load* (4)
200	Floors in dwelling houses tenements, hospital wards, bed rooms and private sitting rooms in hostels and dormitories.	200 kg/m^2 (2 kN/m^2)	
250	Office floor other than entrance halls, floors of light workrooms.	250*–400 kg/m^2 (2.5–4 kN/m^2)	Subject to a min. total load of 2.5 times the values in col. 3 for any given slab panel and 6 times the value in col. 3 for any given beam. This total load shall be asuumed uniformly distributed on the entire area of the slab panel and the entire length of the beam.
300	Floors of banking halls, office entrance halls and reading rooms	300 kg/m^2 (3 kN/m^2)	
400	Shop floors used for display and sale of merchandise ; floors of workrooms generally ; floors of classrooms in schools ; floors of places of assembley with fixed seating ; restaurants ; circulation space in machinery halls ; power stations etc., where not occupied by plant or equipments.	400 kg/m^2 (4 kN/m^2)	
500	Floors of warehouses, workshops, factories and other buildings or parts of buildings of similar categories for light weight loads ; office floors for storage and filling purposes ; floors of places of assembly without fixed seating ; public rooms in hotels, dance halls , waiting halls etc.	500 kg/m^2 (5 kN/m^2)	–
750	Floors of warehouses, workshops, factories and other buildings or parts of buildings of similar categories for medium weight- loads.	750 kg/m^2 (7.5 kN/m^2)	–
1000	Floors of warehouses workshops , factories and other buildings or parts of buildings of similar categories for heavy weight loads, floors of book stores and libraries, roofs and pavement lights over basements projecting under the public footpath.	1000 kg/m^2 (10 kN/m^2)	–

TABLE 1.2 (contd.)

Loading Class No. (1)	*Types of floors* (2)	*Min. Live Load per m^2 of floor area* (3)	*Alternative minimum live load* (4)
Garage, Light	Floors used for garages for vehicles not exceeding 2.5 tonnes (25 kN) gross weight		
	Slab	400 kg/m^2 (4 kN/m^2)	The worst combination of actual wheel loads, whichever is greater.
	Beams	250 kg/m^2 (2.5 kN/m^2)	The worst combination of actual wheel loads, whichever is greater.
Garage, Heavy	Floors used for garages for vehicles not exceeding 4 times gross weight	750 kg/m^2 (7.5 kN/m^2)	Subject to a minimum of one and a half times maximum wheel load but not less than 500 kg (5 kN) considered to be distributed over 75 cm square.
Stairs	Stairs, landing but not considered for class 200 loading but not liable to overcrowding	300 kg/m^2 (c kN/m^2)	Subject to a minimum of 130 kg (1.3 kN) concentrated load at the unsupported end of each step for stairs constructed out of structurally independent cantilever steps.
	Stairs, landing and corridors for class 200 loading but liable to overcrowding, and for all other classes	500 kg/m^2 (5 kN/m^2)	
Balcony	Balconies not liable to overcrowding :		
	For class 200 loading	300 kg/m^2 (3 kN/m^2)	–
	For all other classes	500 kg/m^2 (5 kN/m^2)	–
	Balconies liable to over crowding	500 kg/m^2 (5 kN/m^2)	–

*The lower value of 250 kg/m^2 (2.5 k N/m^2) should be taken where separate storage facilities are provided and the higher value of 400 kg/m^2 (4 kN/m^2) should be taken where such provisions are lacking.

Note 1. In the above table, a reference to a floor includes a reference to any part of that floor, and a reference to 'slabs' includes boarding and beams or ribs, spaced not farther apart than one metre between centres, and a reference to 'beams' means all other beams and ribs.

Note 2. Under loading class No. 250, the reference to 'light work rooms'envisages rooms in which some light machines (for example, sewing machines, used for by milliners or tailors) are operated without a central power-driven unit,that is, the machines are independently operated either by hand or by small motors. Under loading class No. 400, the reference to 'workrooms' generally envisages the installation of machines operated with central power driven unit, with individual machines being belt driven.

Note 3. 'Fixed seating' implies that the removal of the seating and the use of space for other purpose is improbable. The maximum likely load in this case is, therefore, closely controlled.

Note 4. The loading in workshops, warehouses and factories varies considerably and so these loadings under the terms 'light' 'medium' and 'heavy' are introduced in order to allow for more economical designs but the terms have no special meaning in themselves other than the live load for which the relevent floor is designed. It is, however, important, particularly in the case of heavy weight loads, to assess the actual loads to ensure that they are not in excess of 1000 kg/m^2 (10 kN/m^2); in case where they are in excess, the design shall be based on the actual loading.

Note 5. The load classification for stairs, corridors, balconies and landings provide for the fact that these often serve several occupancies and are used for transporting the furniture and goods.

LIVE LOADS ON ROOFS

Table 1.3 gives live loads on flat roofs, sloping roofs and curved roofs.

Roofs of buildings used for promenade or incidental assembly purposes shall be designed for a minimum load of 400 kg/m^2 (4 kN/m^2) or heavier, if required.

Snow Load: If the roof is subjected to snow load, it should be designed for the actual load due to snow or for the live loads specified in Table 1.3, whichever is more severe. Actual load due to snow will depend upon the shape of the roof and its capacity to retain the snow; and each case shall be treated on its own merits. In the absence of any specific information, the loading due to collection of snow may be assumed to be 2.5 kg/m^2 (25 kN/m^2) per centimetre depth of snow.

Loads due to rain: On surfaces whose positioning, shape and drainage system are such as to make accumulation of rainwater possible, loads due to such accumulation of water and the live loads for the roofs as given in Table 1.3 shall be considered separately and the more critical of the two shall be adopted in the design.

TABLE 1.3. LIVE LOADS ON ROOFS (IS : 875-1964)

	Type of roof	*Live load measured on plan*	*Minimum live load measured on plan*
1.	Flat, sloping or ourved with slopes upto and including 10° ::		
	(*a*) Access provided	150 kg/m^2 (1.5 kN/m^2	375 kg (3750 N) uniformly distributed over any span of one metre and 900 kg (9000 N) uniformly distributed over the span in the case of all beams
	(*b*) Access not provided except for maintenance	75 kg/m^2 (0.75 kN/m^2)	190 kg (1900 N) uniformly distributed over any span of one metre width of the roof slab and 450 kg (4500 N) uniformly distributed over the psan in the case of beams
2.	Sloping roof with slope greater than 10°	(*a*) For roof membrane, sheets or purlins – 75 kN/m^2 less 2 kg/m^2 for every degree increase in slope over 10 degrees (*b*) For members supporting the roof membrane and roof purlins such as trusses, beams, girders, etc. - 2/3 of load in (*a*) (*c*) Loads in (*a*) and (*b*) do not include loads due to snow, rain dust collection, etc. and the effects of such loads shall be appropriately considered.	Subject to a minimum of 40 kg/m^2(400 N/m^2)
3.	Curved roof with slope at springing greater 10°	$(75 - 345\gamma^2)$ kg/m^2 where $\gamma = \frac{h}{l}$ h = height of the highest point of the structure measured from its springings, and l = chord length of the roof if singly curved and shorter of the two sides if doubly curved.	Subject to a minimum of 40 kg/m^2

Note . For special types of roofs with highly permeable and absorbent material, the contingency of roof material increasing in weight due to absorption of moisture shall be provided for.

[1kg /m^2 = 10N/m^2 = 0.01 kN/m^2]

3. Wind Loads

Wind is the air in motion relative to the surface of the earth. Since vertical components of atmospheric motion are relatively small, specially near the surface of the earth, the term 'wind' denotes almost exclusively the horizontal wind. Wind pressure, therefore, acts horizontally on the exposed vertical surfaces of walls, columns etc, and inclined roof of the structure. *Wind pressure or wind loading* is expressed in terms of a *basic pressure p* which is an equivalent static pressure in the direction of flow of wind. The *basic wind pressure p* should be decided on the basis of local meteorological data and local conditions, such as characteristics and location of structure and characteristics and duration of wind flow. In absence of any other data, the following relationship may be obtained by the expression.

$$p = kV^2 \qquad \text{...(1.1)}$$

where p = wind pressure, kg/m^2

V = wind velocity, km/hours

and k = co-efficient, the value of which depends upon a number of factors, such wind speed, the type, proportion and shape of structure and the temperature of air. National Building Code has adopted $k = 0.006$ for the preparation of wind pressure maps. For detailed study, reference may be made to these maps. These maps and the associated charts give the variation in wind pressure with height.

For structures of various plan shapes other than rectangular plan shape, the external pressures acting on the projected area in the plane perpendicular to the wind, should be the product of the basic pressure (p) given above and the shape factors given in Table 1.4.

TABLE 1.4. PLAN SHAPE FACTORS

Plan shape of the structure	*Factor*		
	Ratio of height to base width 0–4	*Ratio of height to base width 4–8*	*Ratio of height to base width 8 or over*
1. Circular	0.7	0.7	0.7
2. Octagonal	0.8	0.9	1.0
3. Square (wind perpendicular to diagonal	0.8	0.9	1.0
4. Square (wind perpendicular to face)	1.0	1.15	1.3

The effect of wind shall be calculated on the basis of basic wind pressure(p) for the entire height of the building and any projection

thereof, having due regard to the level of mean retarding surface and variation in wind pressure with height. In making calculations, due regard should be given to shape factor and internal air pressure.

The stability calculations of the building as a whole shall be done considering the combined effect, as well as separate effects of imposed loads and wind loads on vertical surfaces, roofs and any other part of the building above the general roof level.

Calculations shall be made for the effect of wind on the design of individual components of the building. Where, however, adequate stiffening is provided by walls, or floors and walls, calculations for the effects of wind, except in regard to wind panels, roofs and foundations, need not be made on:

(*a*) A building or part of a building of which the height *h* does not exceed twice the effective widths,

(*b*) a section adjoining two parts of an adequately stiffened building, if height of the section exceeds twice its width but the length of the section does not exceed four times its width, and

(*c*) a wing of such a building if it does not project more than twice its own width.

In the case of sloped roofs, the wind pressure varies with the degree of slope. The slope of the roof towards the wind direction is known as the *windward slope*, while the slope to the other side of the wind direction is known as the *leeward slope*. For flat and pitched roofs, the wind pressures normal to the surface, due to wind blowing at right angles to the ridge, in terms of *basic wind pressure*(p) are given in Table 1.5.

TABLE 1.5. WIND PRESSURE ON ROOFS (WIND NORMAL TO RIDGE)

S.No.	*Slope of roof on windward side* (α)	*External wind pressure*	
		windward slope	*Leeward slope*
1.	0°	− 1.00 p	− 0.50 p
2.	10°	− 0.70 p	− 0.50 p
3.	20°	− 0.40 p	− 0.50 p
4.	30°	− 0.10 p	− 0.50 p
5.	40°	+ 0.10 p	− 0.50 p
6.	50°	+ 0.30 p	− 0.50 p
7.	60°	+ 0.40 p	− 0.50 p
8.	70°	+ 0.50 p	− 0.50 p
9.	80°	+ 0.50 p	− 0.50 p
10.	90°	+ 0.50 p	− 0.50 p

In Table 1.5, plus (+) means an external positive pressure on the roof normal to the surface of the roof and minus (–) means an external suction on the roof normal to the surface of the roof.

2

Foundations - 1 : General

2.1. INTRODUCTION

Every building consists of two basic components : the *super-structure* and the *substructure* or *foundations*. The super-structure is usually that part of the building which is above ground, and which serves the purpose of its intended use. The *substructure* or *foundations* is the lower portion of the building, usually located below ground level, which transmits the load of the super-structure to the sub-soil. *A foundation is therefore that part of the structure which is in direct contact with the ground to which the loads are transmitted.* The soil which is located immediately below the *base of the foundation is called the sub-soil or foundation soil* , while the lowermost portion of the foundation which is in direct contact with the sub-soil is called the *footing*.

The basic function of a foundation is to transmit the dead loads, super-imposed loads (or live loads) and wind loads from a building to the soil on which the building rests, in such a way that (*a*) settlements are within permissible limits, without causing cracks in the super-structure, and (*b*) the soil does not fail. When loads are transmitted to the sub-soil, it settles. If this *settlement is slight* and *uniform* throughout, no damage will be caused to the building. But if the settlement is *excessive or unequal*, serious damage may result in the form of cracked walls, distorted doors and window openings, cracked lintels, walls thrown out of plumb etc., and sometimes the complete collapse of the building.

The foundation is thus the most important part of a building. Since it remains below the ground level, the signs of failure of foundation are not noticeable till it has already affected the building. A foundation should be sufficiently strong to prevent excessive settlement as well as unequal settlement. *Unequal settlement or differential settlement* may be caused by (*i*) weak sub-soils, such as made up ground (*ii*) shrinkable and expansive soils (such as clay), (*iii*) frost action, (*iv*)

movement of ground water, and uplift pressure, (*v*) excessive vibrations, due to traffic, machinery etc., (*vi*) slow consolidation of saturated clays, and (*vii*) slipping of strata on sloping sites. When designing the foundations, therefore, the above factors must be taken into account.

2.2 FUNCTIONS OF FOUNDATIONS

Foundations serve the following purposes :

1. ***Reduction of load intensity.*** Foundations distribute the loads of the super-structure, to a *larger area* so that the intensity of the load at its base (*i.e.* total load divided by the total area) does not exceed the *safe bearing capacity* of the sub-soil. In the case of deep foundations, it transmits the super-imposed loads to the sub-soil both through side friction as well as through end bearing.

2. ***Even distribution of load.*** Foundations distribute the non-uniform load of the super-structure *evenly* to be sub-soil. For example, two columns carrying unequal loads can have a combined footing which may transmit the load to sub-soil evenly with uniform soil pressure. Due to this, unequal or differential settlements are minimised.

3. ***Provision of level surface.*** Foundations provide levelled and hard surface over which the super-structure can be built.

4. ***Lateral stability.*** It anchors the super-structure to the ground, thus imparting lateral stability to the super-structure. The stability of the building, against sliding and overturning, due to horizontal forces(such as wind, earthquake etc.) is increased due to foundations.

5. ***Safety against undermining.*** It provides the structural safety against undermining or scouring due to burrowing animals and flood water.

6. ***Protection against soil movements.*** Special foundation measures prevents or minimises the distress (or cracks) in the super-structure, due to expansion or contraction of the sub-soil because of moisture movement in some problematic soils.

2.3. ESSENTIAL REQUIREMENTS OF A GOOD FOUNDATION

Foundations should be constructed to satisfy the following requirements :

1. The foundations shall be constructed to *sustain* the dead and imposed loads and to *transmit* these to the sub-soil in such a way that pressure on it will not cause settlement which would impair the stability of the building or adjoining structures.

2. Foundation base should be rigid so that differential settlements are minimised, specially for the case when super-imposed loads are not evenly distributed.

3. Foundations should be taken sufficiently deep to guard the

building against damage or distress caused by swelling or shrinkage of the sub-soil.

4. Foundations should be so located that its performance may not be affected due to any unexpected future influence.

2.4. TYPES OF FOUNDATIONS

Foundations may be broadly classified under two heads :

(*a*) Shallow Foundations

(*b*) Deep Foundations.

According to Terzaghi, a foundation is *shallow* if its depth is equal to or less than its width. In case of *deep foundations,* the depth is equal to or greater than its width.

(A) SHALLOW FOUNDATIONS

From the point of view of design, shallow foundations may be of the following types:

1. Spread footings.
2. Combined footings
3. Strap footings.
4. Mat foundation.

Various types of *shallow foundations* are shown in Fig. 2.1.

. A brief description of these is given below. Details about the design requirements etc. are discussed in Chapter 3.

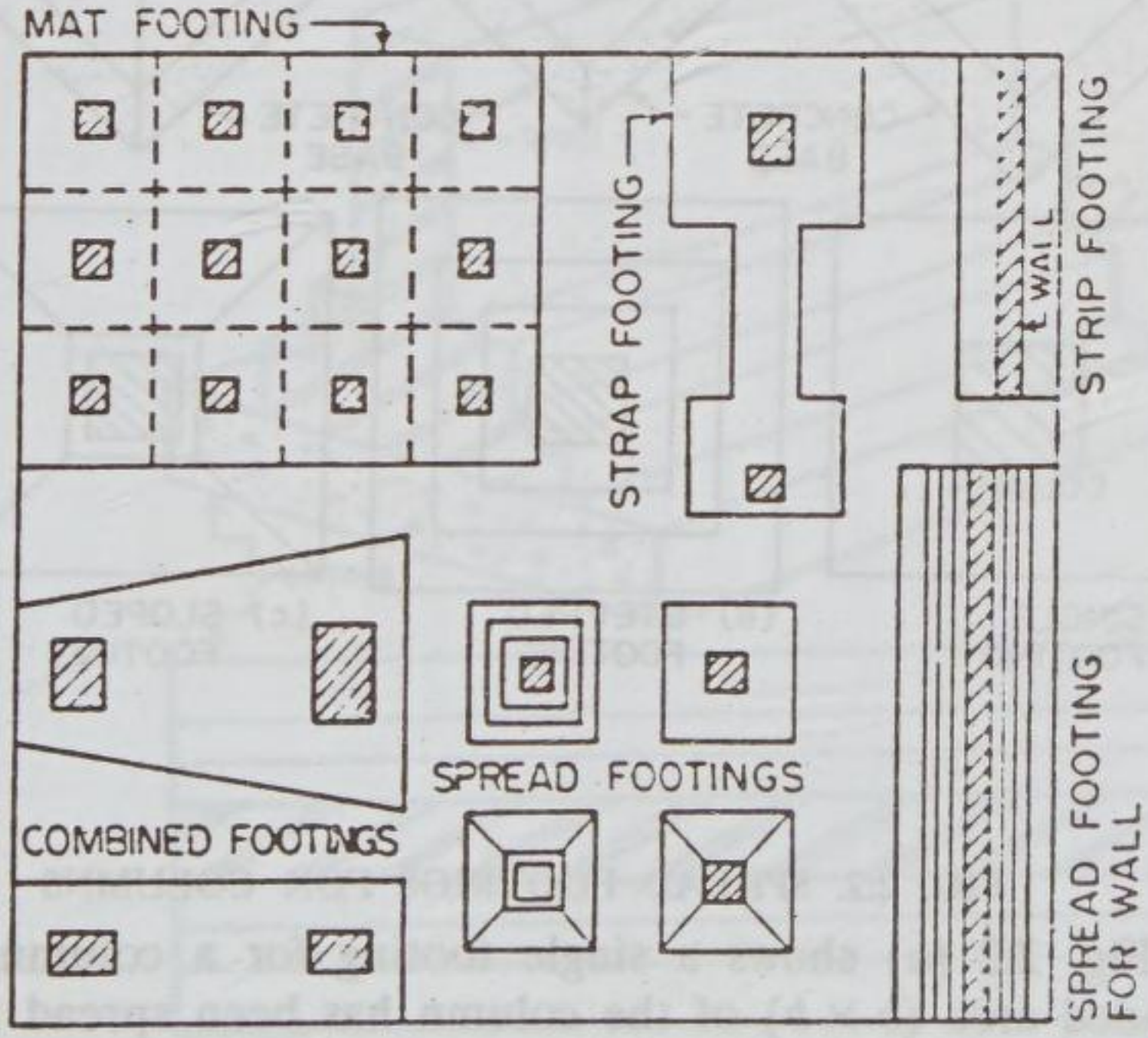


FIG. 2.1. VARIOUS TYPES OF SHALLOW FOUNDATIONS.

1. Spread Footings

Spread footings are those which spread the super-imposed load of wall or column over a larger area. Spread footings support

either a column or wall. Spread footings may be of the following kinds :

(*i*) Single footing [Fig. 2.2(*a*)] for a column
(*ii*) Stepped footing [Fig. 2.2(*b*)] for a column
(*iii*) Sloped footing [Fig. 2.2(*c*)] for a column
(*iv*) Wall footing without step [Fig. 2.3(*a*)]
(*v*) Stepped footing for wall [Fig. 2.3(*b*)]
(*vi*) Grillage foundation [Fig. 2.4)].

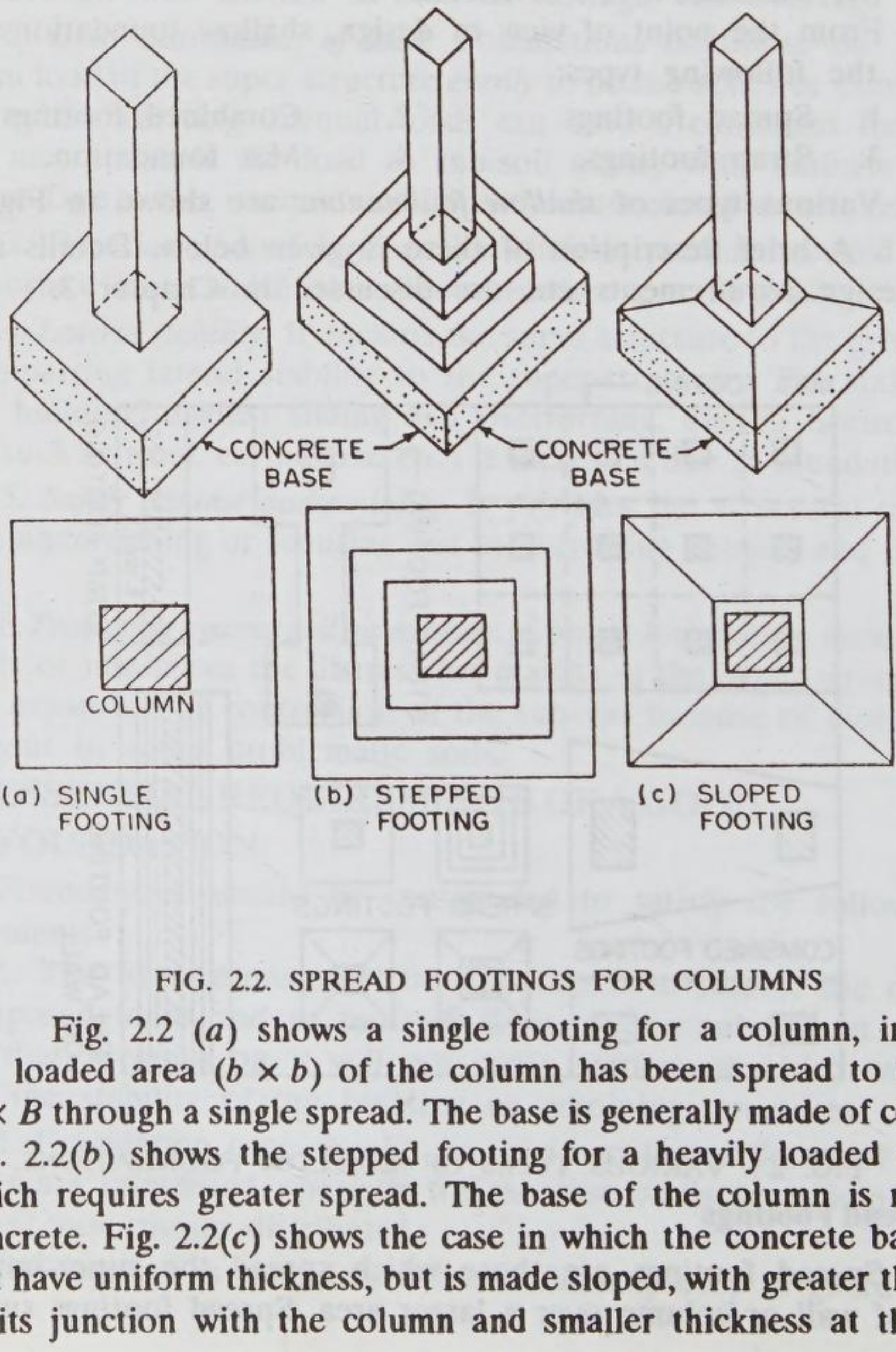


FIG. 2.2. SPREAD FOOTINGS FOR COLUMNS

Fig. 2.2 (*a*) shows a single footing for a column, in which the loaded area ($b \times b$) of the column has been spread to the size $B \times B$ through a single spread. The base is generally made of concrete. Fig. 2.2(*b*) shows the stepped footing for a heavily loaded column, which requires greater spread. The base of the column is made of concrete. Fig. 2.2(*c*) shows the case in which the concrete base does not have uniform thickness, but is made sloped, with greater thickness at its junction with the column and smaller thickness at the ends.

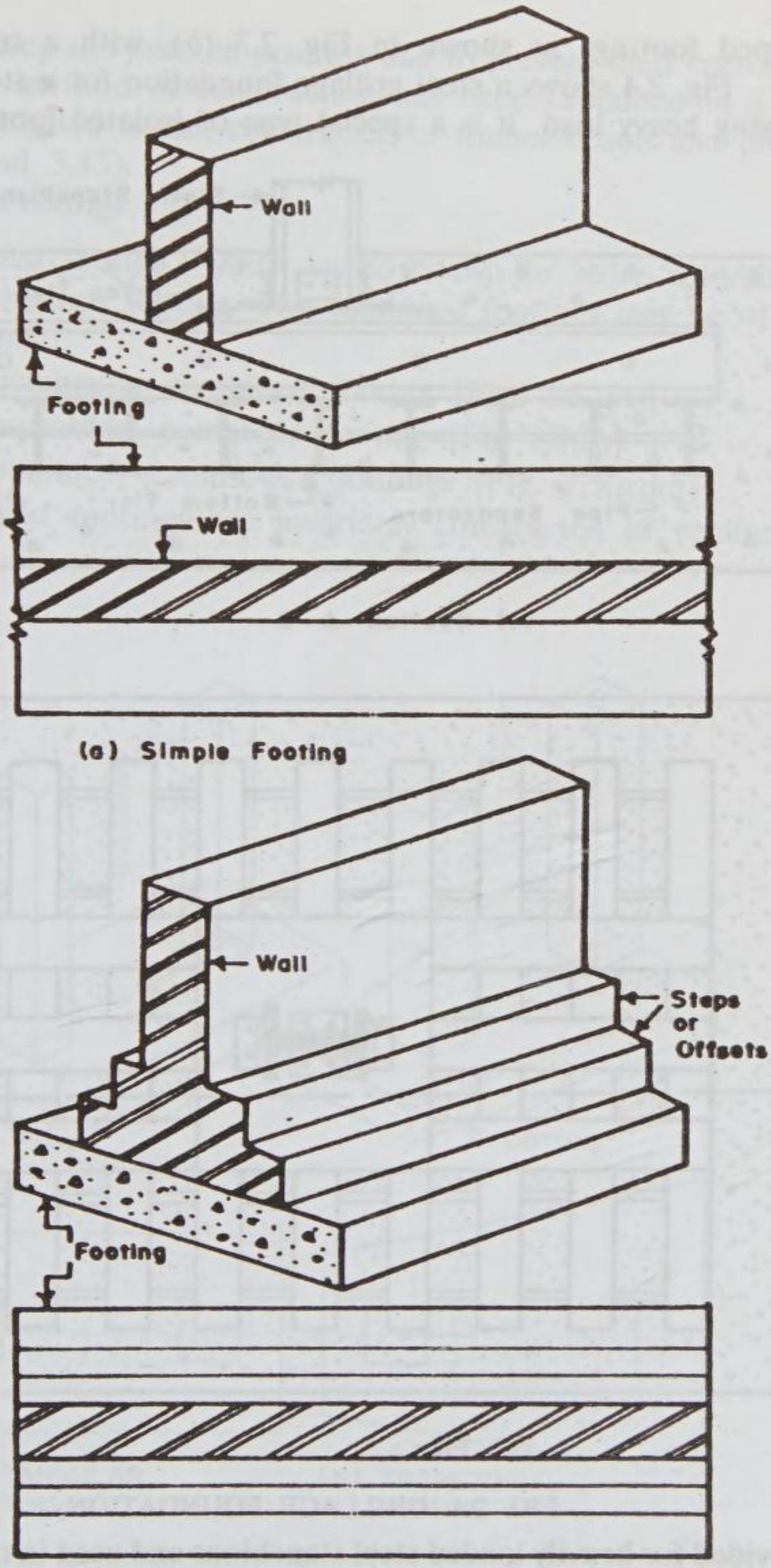
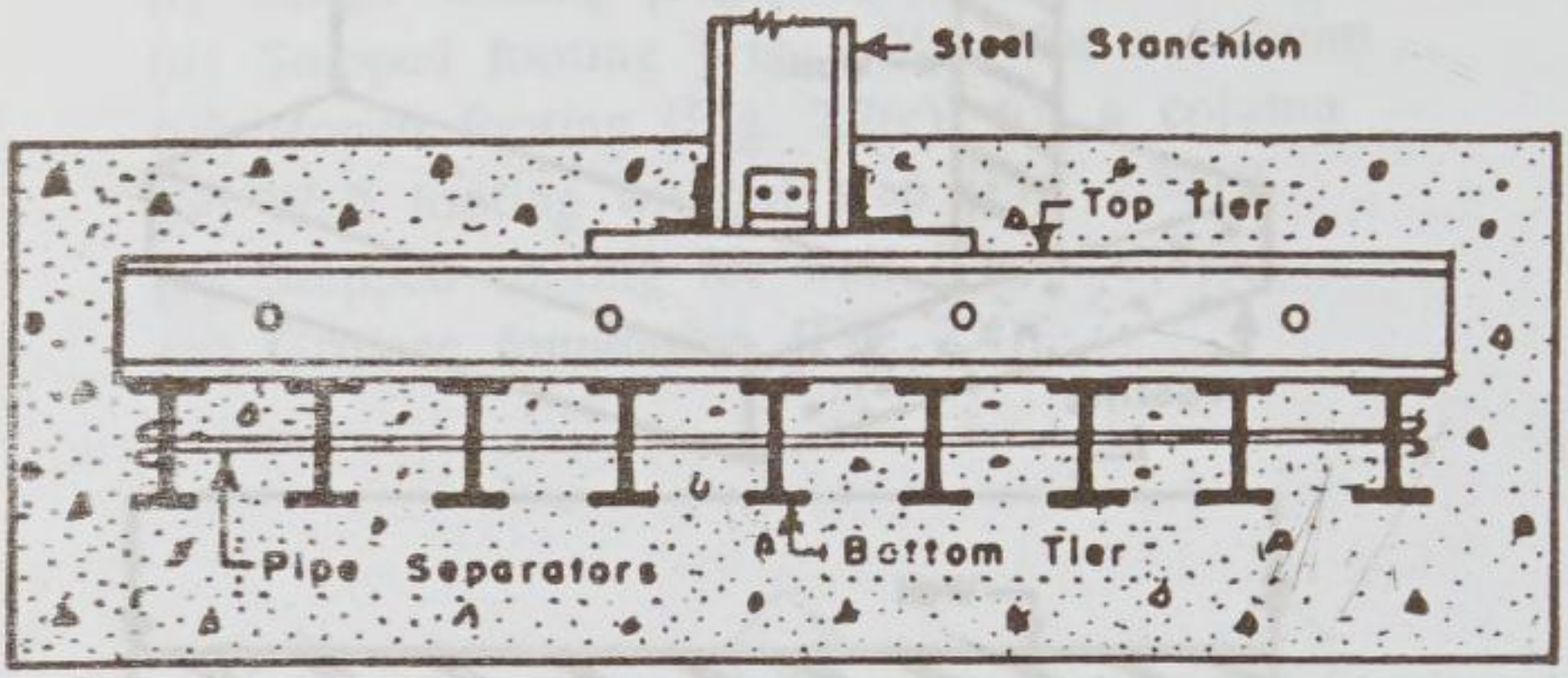


FIG. 2.3. SPREAD FOOTING FOR WALLS : STRIP FOOTING.

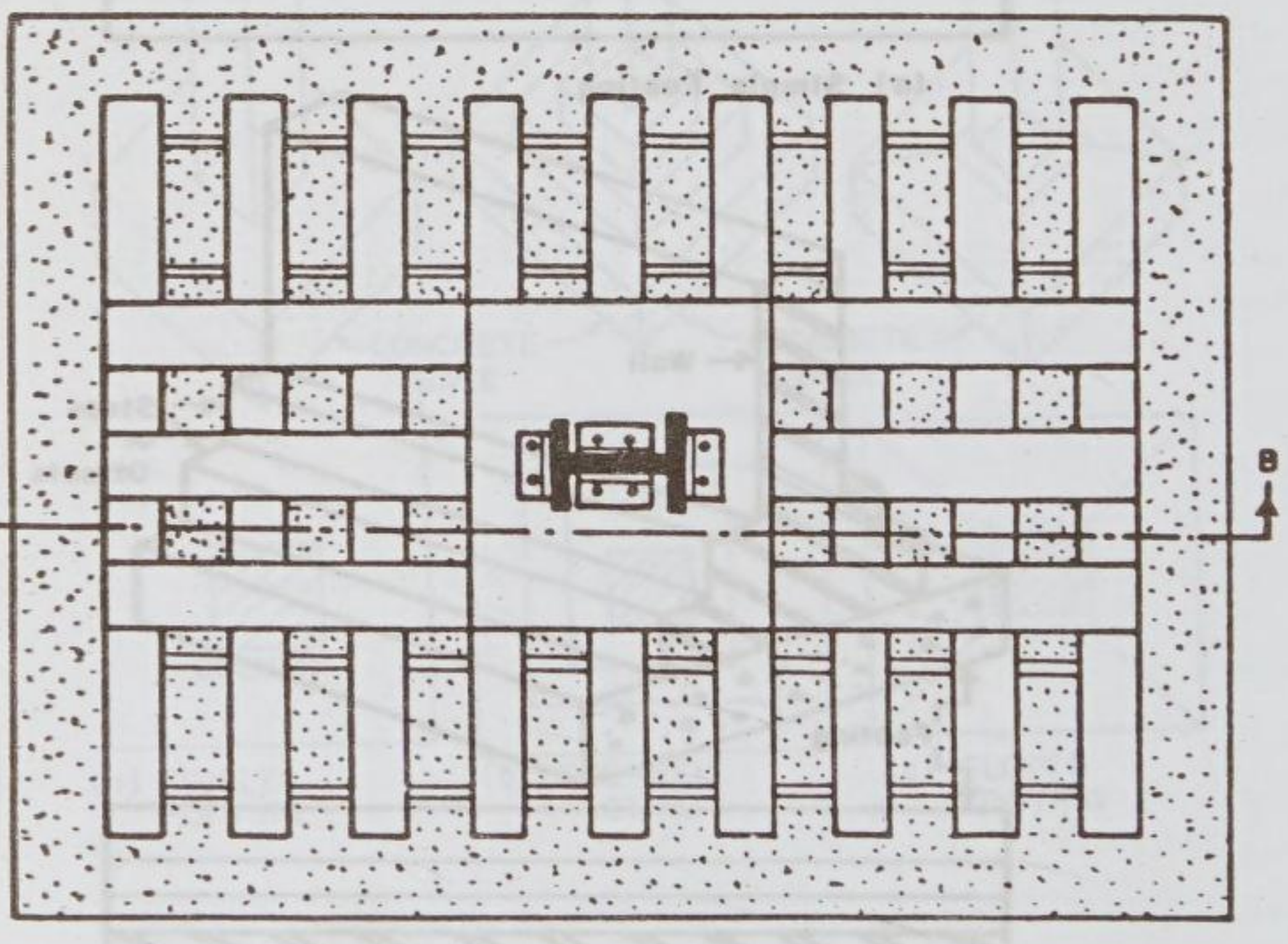
Fig. 2.3 (*a*) shows the spread footing for a wall, consisting of concrete base without any steps. Usually, masonry walls have

stepped footings as shown in Fig. 2.3 (*b*), with a concrete base.

Fig. 2.4 shows a steel grillage foundation for a steel stanchion carrying heavy load. It is a special type of isolated footing generally



(a) Section A-B



(b) Plan

FIG. 2.4. GRILLAGE FOUNDATION.

provided for heavily loaded steel stanchions and used in these locations where bearing capacity of soil is poor. The depth of such a foundation is limited to 1 to 1.5 m. The load of the stanchion is distributed or spread to a very large area by means of two or more tiers of rolled steel joints, each layer being laid at right angles to the layer below it. Both the tiers of the joists are then embeded in cement

concrete to keep the joists in position and to prevent their corrosion. The detailed method of construction has been explained in § 3.6. Grillage foundation is also constructed of timber beams and planks (Fig. 3.12 and 3.13).

2. Combined Footings

A spread footing which supports two or more columns is termed as combined footing. The combined footings may be of the following kinds:

(*i*) Rectangular combined footing [Fig. 2.5 (*a*)]

(*ii*) Trapezoidal combined footing [Fig 2.5(*b*)]

(*iii*) Combined column-wall footings [Fig. 2.5(*a*),(*b*)]

Combined footings are invariably constructed of reinforced concrete.

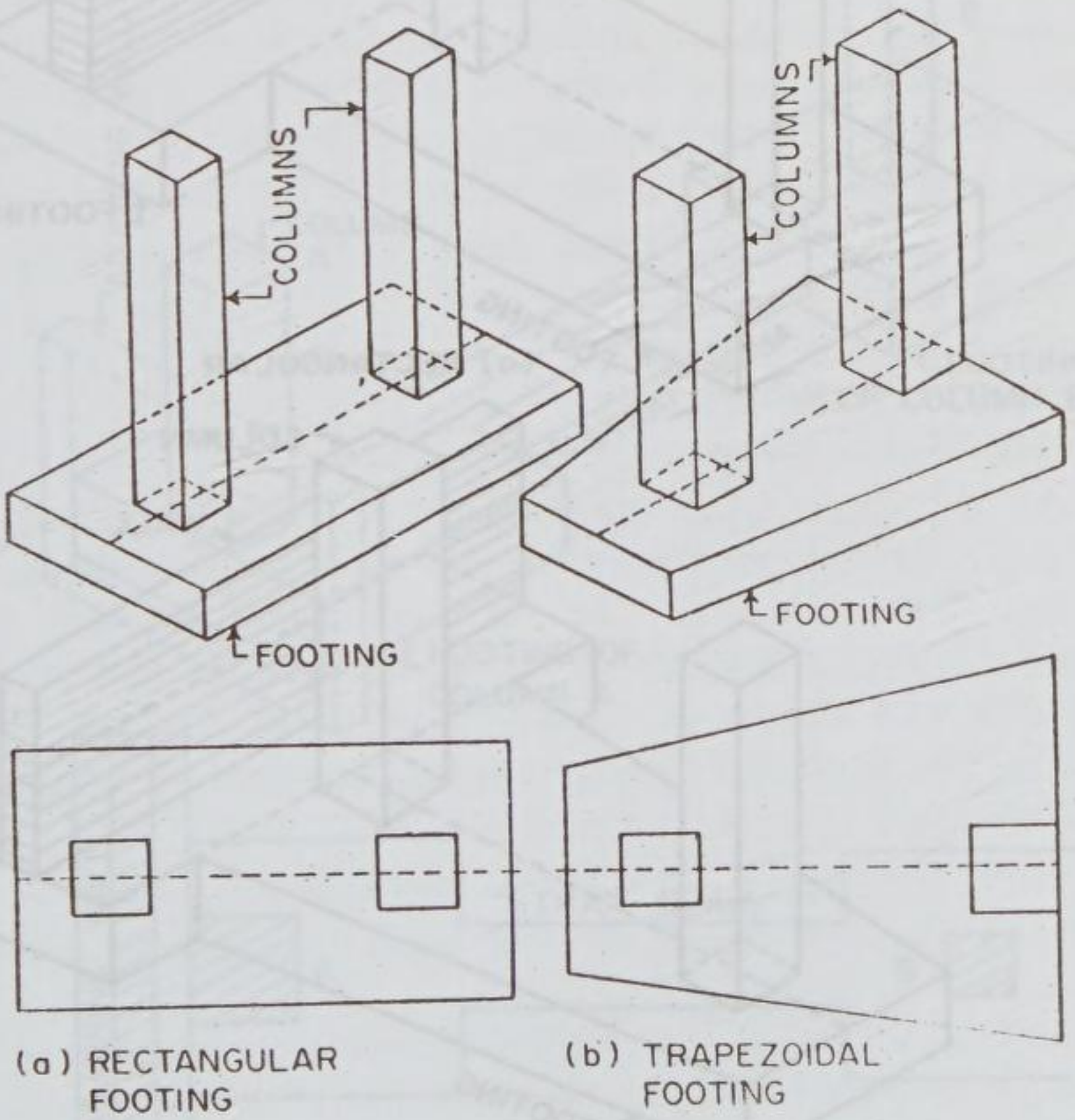


FIG. 2.5. COMBINED FOOTINGS FOR COLUMNS.

The combined footing for columns will be rectangular in shape if they carry equal loads. The design of rigid rectangular combined

footing should be done in such a way that centre of gravity of column loads coincide with the centroid of the footing area. If the columns carry unequal loads, the footing is of trapezoidal shape, as shown in Fig. 2.5 (*b*).

Sometimes, it may be required to provide a combined footing for columns and a wall. Such combined footings are shown in Fig

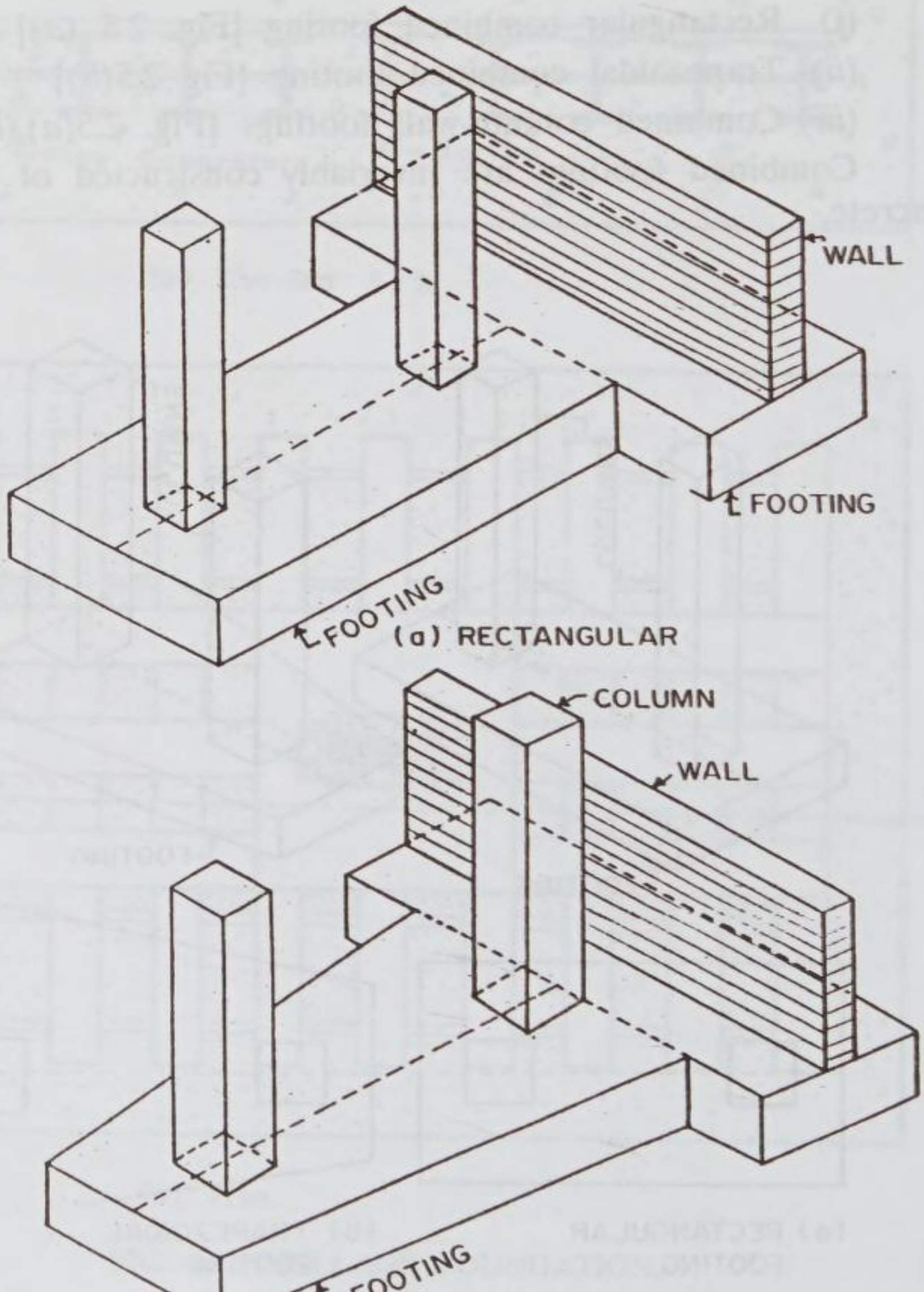


FIG. 2.6. COMBINED FOOTINGS FOR COLUMNS AND WALL.

2.6 (*a*) [when the columns carry equal loads] and in Fig. 2.6 (*b*) [when the columns carry unequal loads] . The design principles of these footings have been discussed in Chapter 3.

3. Strap Footings

If the independent footings of two columns are connected by a beam, it is called a *strap footing.* A strap footing may be used where the distance between the columns is so great that a combined trapezoidal footing becomes quite narrow, with high bending moments. In that case, each column is provided with its independent footings and a beam is used to connect the two footings. The strap beam does not remain in contact with soil, and thus does not transfer

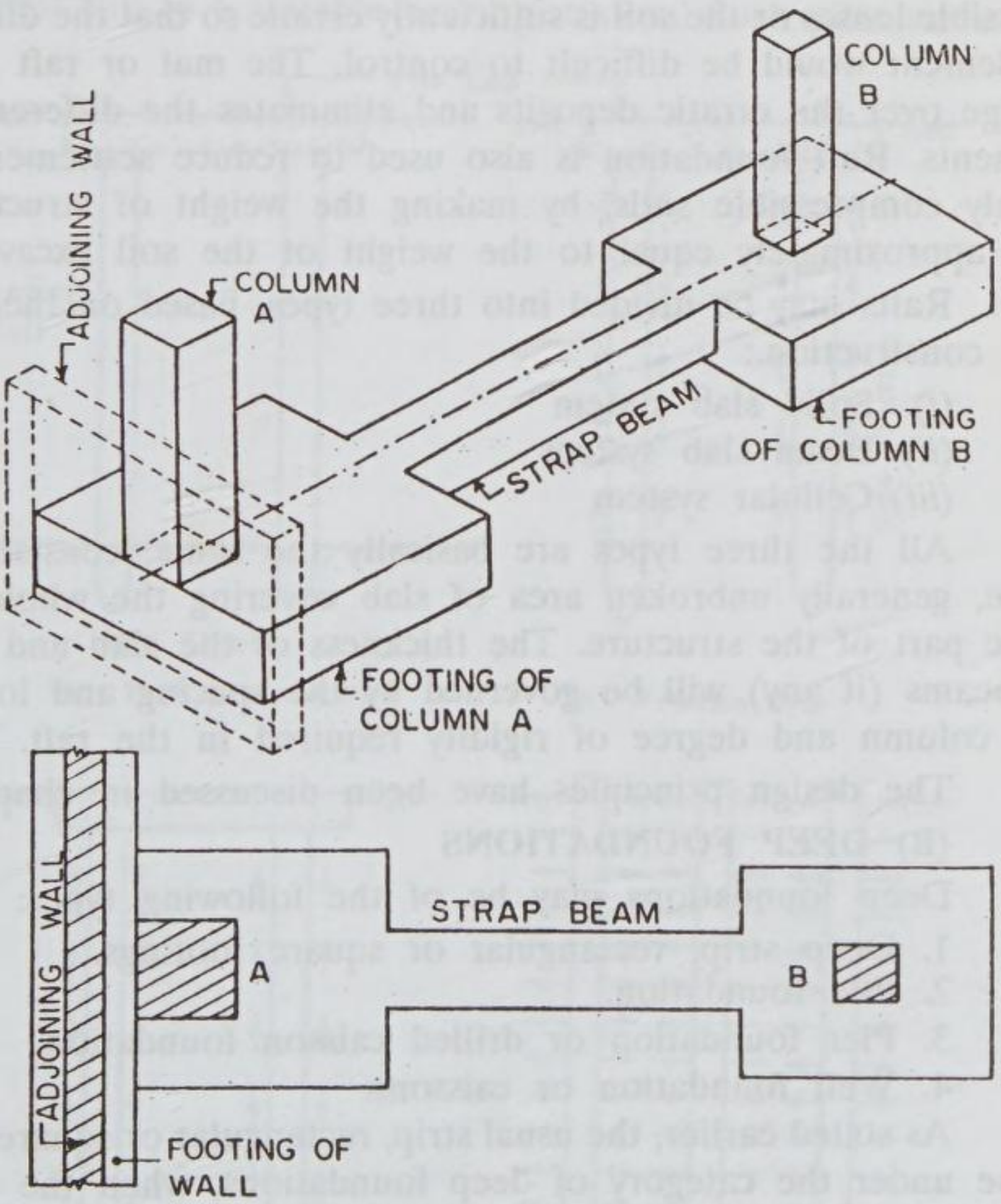


FIG. 2.7. STRAP FOOTING.

any pressure to the soil. The strap, assumed to be infinitely stiff, serves to transfer the column loads on to the soil with equal and uniform soil pressure under both footings.

Fig. 2.7 shows the strap footing for two columns *A* and *B*. Column *A* is so near to an existing wall that the footing of the wall does not permit the independent footing of column *A* to spread out towards the wall, though it has freedom in other directions.

4. Mat Foundation (Raft Foundation)

A raft or mat is a combined footing that covers the entire area beneath a structure and supports all the walls and columns. When the allowable soil pressure is low, or the building loads are heavy, the use of spread footings would cover more than one half the area and it may prove more economical to use *mat or raft* foundation. They are also used where the soil mass contains compressible lenses or the soil is sufficiently erratic so that the differential settlement would be difficult to control. The mat or raft tends to bridge over the erratic deposits and eliminates the differential settlements. Raft foundation is also used to reduce settlement above highly compressible soils, by making the weight of structure and raft approximately equal to the weight of the soil excavated.

Rafts may be divided into three types, based on their design and construction.:

(*i*) Solid slab system
(*ii*) Beam slab system
(*iii*) Cellular system

All the three types are basically the same, consisting of a large, generally unbroken area of slab covering the whole or the large part of the structure. The thickness of the slab and the size of beams (if any) will be governed by the spacing and loading of the column and degree of rigidity required in the raft.

The design principles have been discussed in chapter 3.

(B) DEEP FOUNDATIONS

Deep foundations may be of the following types:

1. Deep strip, rectangular or square footings.
2. Pile foundation.
3. Pier foundation or drilled caisson foundation.
4. Well foundation or caissons.

As stated earlier, the usual strip, rectangular or square footings come under the category of deep foundations, when the depth of the foundation is more than the width of the footing. Well foundations are generally adopted for bridge piers etc. and not for building foundations.

Pile foundation

Pile foundation is that type of deep foundation in which the

loads are taken to a low level by means of vertical members which may be of timber, concrete or steel. Pile foundation may be adopted (*i*) instead of a raft foundation where no firm bearing strata exists at any reasonable depth and the loading is uneven, (*ii*) when a firm bearing strata does exist but at a depth such as to make strip or spread footing uneconomical, and (*iii*) when pumping of sub-soil water would be too costly or timbering to excavations too difficult to permit the construction of normal foundations.

Piles used for building foundation may be of four types :

(*i*) End bearing pile [Fig. 2.8 (*a*)]

(*ii*) Friction pile [Fig. 2.8 (*b*)]

(*iii*) Combined end bearing and friction pile [Fig. 2.8 (*c*)] and

(*iv*) Compaction piles [Fig. 2.8 (*d*)]

End bearing piles [Fig. 2.8 (*a*)] are used to transfer load through water or soft soil to a suitable bearing stratum. Such piles are used

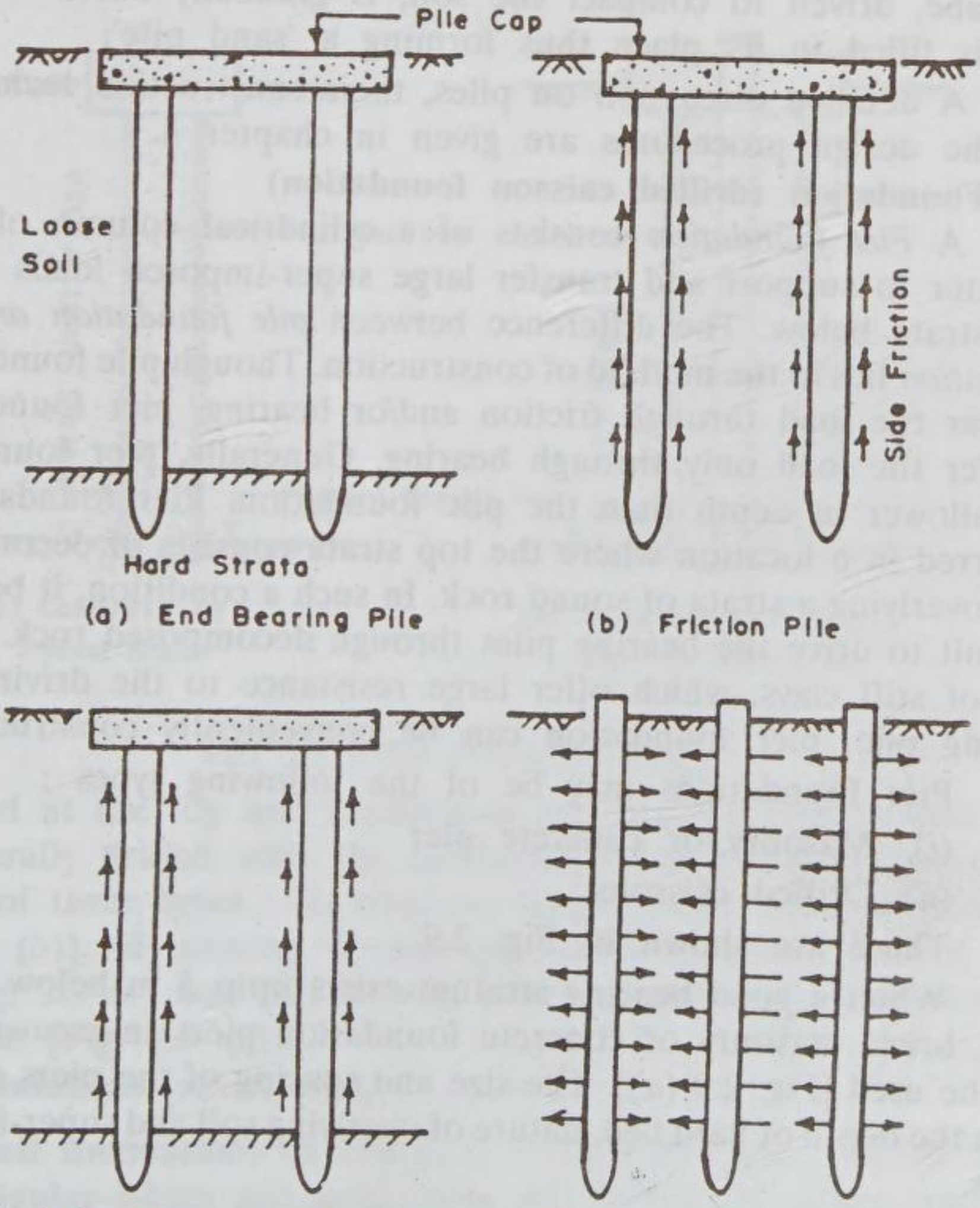


FIG. 2.8. PILE FOUNDATIONS

to carry heavy loads safely to hard strata. Multi-storeyed buildings are invariably founded on end bearing piles, so that the settlements are minimised.

Friction piles [Fig. 2.8 (*b*)] are used to transfer loads to a depth of a friction-load-carrying material by means of skin friction along the length of the pile. Such piles are generally used in granular soil where the depth of hard stratum is very great.

Fig. 2.8 (*c*) shows a pile which transfers the super-imposed load both through side friction as well as end bearing. Such piles are more common, specially when the end bearing piles pass through granular soils.

Compaction piles [Fig. 2.8 (*d*)] are used to compact loose granular soils, thus increasing their bearing capacity. The compaction piles themselves do not carry a load. Hence they may be of weaker material (such as timber, bamboo sticks etc.)–sometimes of sand only. The pile tube, driven to compact the soil, is gradually taken out and sand is filled in its place thus forming a 'sand pile'.

A detailed discussion on piles, their construction techniques and the design procedures are given in chapter 4.

Pier Foundation (drilled caisson foundation)

A *Pier foundation* consists of a cylindrical column of large diameter to support and transfer large super-imposed loads to the firm strata below. The difference between *pile foundation and pier foundation* lies in the method of construction. Though pile foundations transfer the load through friction and/or bearing, pier foundations transfer the load only through bearing. Generally, pier foundation is shallower in depth than the pile foundation. Pier foundation is preferred in a location where the top strata consists of decomposed rock overlying a strata of sound rock. In such a condition, it becomes difficult to drive the bearing piles through decomposed rock. In the case of stiff clays, which offer large resistance to the driving of a bearing pile, pier foundation can be conveniently constructed.

Pier foundations may be of the following types :

(*i*) Masonry or concrete pier

(*ii*) Drilled caissons.

These are shown in Fig. 2.9.

When a good bearing stratum exists upto 5 m below ground level, brick, masonry or concrete foundation piers in *excavated pits* may be used [Fig. 2.9 (*a*)]. The size and spacing of the piers depends upon the depth of hard bed, nature of overlying soil and super-imposed loads.

The terms *drilled caissons, foundation pier* or *sub-pier* are interchangeably used by engineers to denote a cylindrical foundation. A *drilled caisson* is largly a compressed member subjected to an

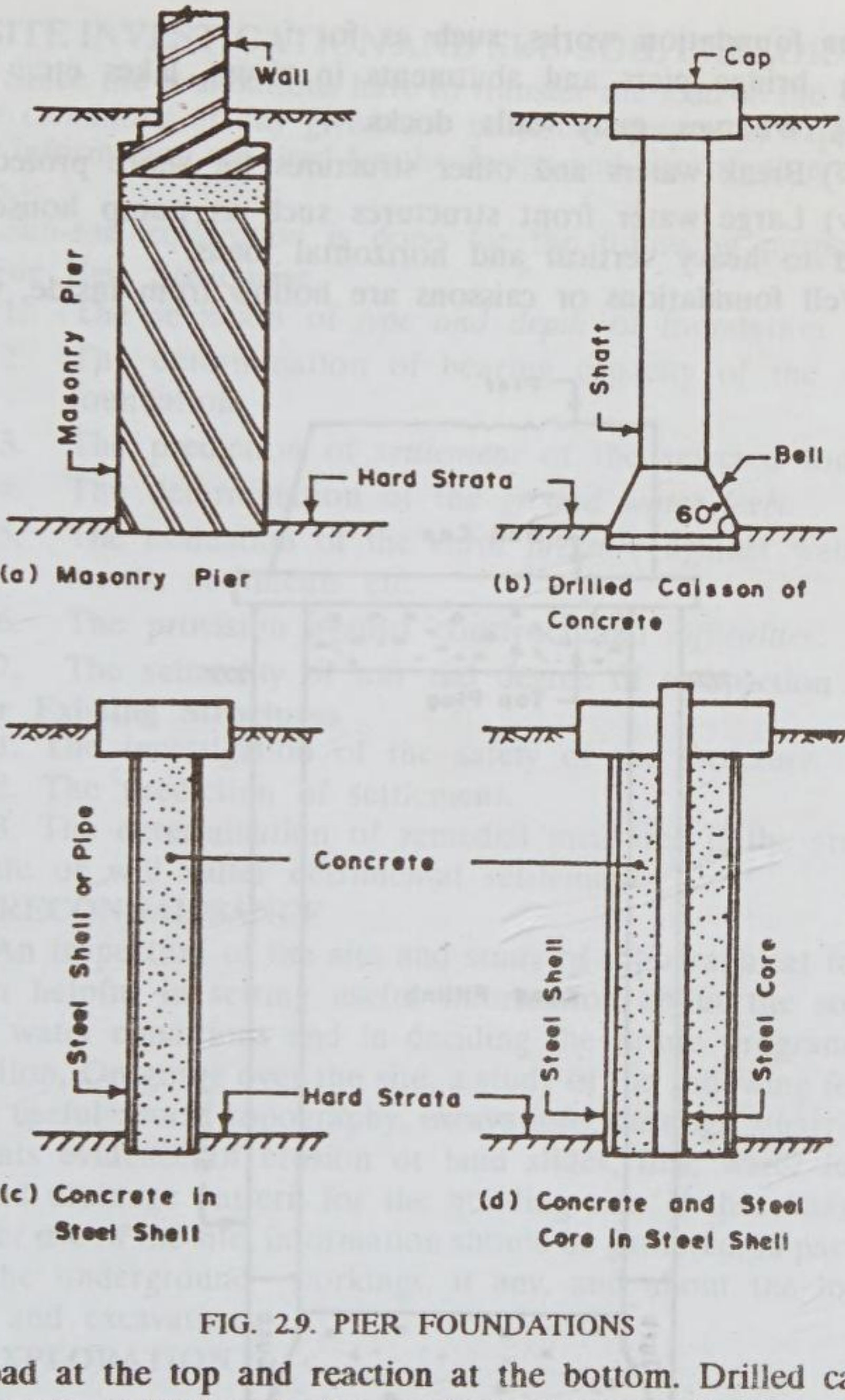


FIG. 2.9. PIER FOUNDATIONS

axial load at the top and reaction at the bottom. Drilled caissons are generally drilled with the mechanical means. Drilled caissons may be of three types : (*i*) concrete caisson with enlarged bottom [Fig. 2.9 (*b*)], (*ii*) caisson of steel pipe with concrete filled in the pipe [Fig. 2.9 (*c*) and (*iii*) caisson with concrete and steel core in steel pipe [Fig. 2.9 (*d*)].

Well Foundations (or caissons)

Well foundations or caissons are box like structure–circular or rectangular–which are *sunk* from the surface of either land or water to the desired depth. They are much large in diameter than the *peir foundations* or *drilled caissons*. Caisson foundations are used

for major foundation works, such as for :

(*i*) bridge piers and abutments in rivers, lakes etc.

(*ii*) Wharves, quay walls, docks.

(*iii*) Break waters and other structures for shore protections.

(*iv*) Large water front structures such as pump houses, subjected to heavy vertical and horizontal loads.

Well foundations or caissons are hollow from inside, which

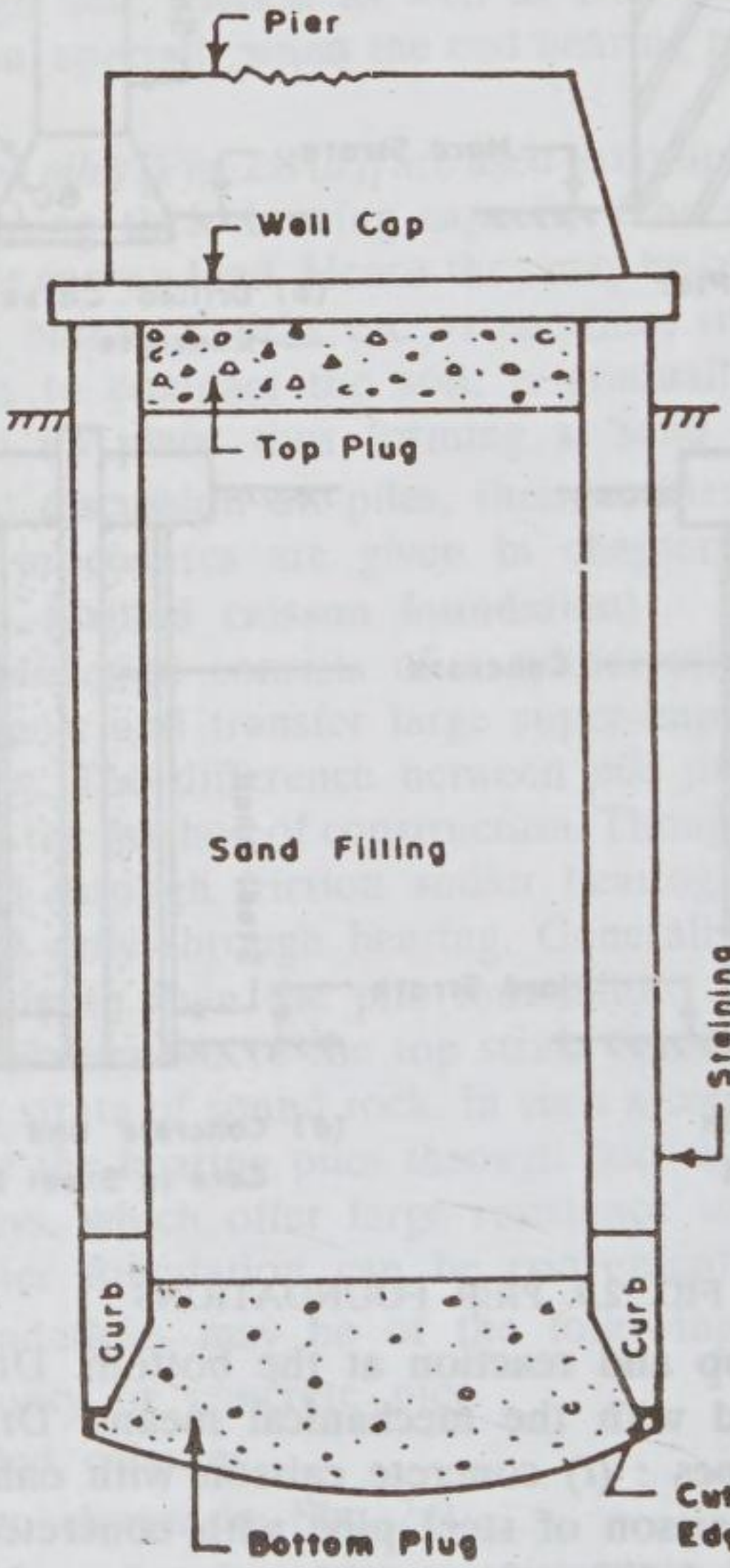


FIG. 2.10. WELL FOUNDATION.

may be filled with sand, and are plugged at the bottom. The load is transferred through the perimeter wall, called steining (Fig. 2.10).

Well foundations are not used for buildings.

2.5. SITE INVESTIGATION AND SUB-SOIL EXPLORATION

Since the foundations have to transfer the load to the sub-soil, *surface conditions* at any given site must be adequately explored to obtain information required for the design and construction of foundations.

Sub-soil exploration is done for the following purposes:

(*a*) For New Structures

1. The selection of *type and depth* of foundation.
2. The determination of bearing capacity of the selected foundation.
3. The prediction of *settlement* of the selected foundation.
4. The determination of the *ground water level.*
5. The evaluation of the *earth pressure* against walls, basements, abutments etc.
6. The provision against constructional *difficulties.*
7. The suitability of soil and degree of compaction of soil.

(*b*) For Existing Structures

1. The investigation of the safety of the structure.
2. The prediction of settlement.
3. The determination of remedial measures if the structure is unsafe or will suffer detrimental settlement.

SITE RECONNAISSANCE

An inspection of the site and study of topographical features is often helpful in setting useful information about the soil and ground water conditions and in deciding the future programme of exploration. On going over the site, a study of the following features may be useful : local topography, excavations, cuttings, quarries, escarpments evidence of erosion or land slides, fills, water level in wells and drainage pattern for the building site. If there has been an earlier use of the site, information should be gathered, in particular about the underground workings, if any, and about the location of fills and excavations.

SITE EXPLORATION

The object of the site exploration is to provide reliable, specific and detailed information about the soil and ground water conditions of the site which may be required for a safe and economic design of foundations. For this purpose, an exploration of the region likely to be affected by the proposed works should yield precise information about the following :

(*i*) the order of occurrence and extent of soil and rock strata.

(*ii*) the nature and engineering properties of the soil and rock formation, and

(*iii*) the location of ground water and its variation.

Depth of exploration

Exploration, in general, should be carried out to a depth upto which the increase in pressure due to structural loading is likely to cause perceptible settlement or shear failure of foundations. Such a depth, known as *significant depth*, depends upon the type of structure, its weight, size, shape and disposition of the loaded areas, and the soil profile and its properties. The significant depth may be assumed to be equal to one-and-a-half to two times the width (smaller of the lateral dimension) of the loaded area.

The depth of exploration at the start of the work may be decided according to the following guide rules, which may need modification as exploration proceeds :

1. *Isolated spread footing or raft*: One and a half times the width.

2. *Adjacent footings with clear spacing less than twice the width*: One and a half times the length.

3. *Pile foundation*: 10 to 30 metres, or more, or at least one and a half times the width of the structure.

4. *Base of the retaining wall*: One and a half times the base width or one and a half times the exposed height of face of wall, whichever is greater.

5. *Floating basement*: Depth of construction.

6. *Weathering considerations* : 1.5 m in general and 3.5 m in black cotton soils.

National Building Code of India (SP:7-1983) suggests that *normally* the depth of exploration should be one and half times the estimated width (lower dimension) of the footing, single or combined, from the *base level* of the foundation; but in weak soils, the exploration should be continued to a depth at which the loads can be carried by the stratum in question without undesirable settlement or shear failure. In any case, the depth to which weathering processes affect the soil should be regarded as a minimum depth for the exploration of sites and this should be taken as 1.5 metres. But where industrial processes affect the soil characteristics, this depth may be more.

2.6 METHODS OF SITE EXPLORATION

The various methods of site exploration may be grouped as follows:

1. Open excavations
2. Borings
3. Sub-surface soundings
4. Geo-physical methods.

1. OPEN EXCAVATION (OPEN TRIAL PITS)

Trial pits are the cheapest method of exploration in shallow deposits, since these can be used in all types of soils. In this method, pits are excavated at the site, exposing the sub-soil surface thoroughly. Soil samples are collected at various levels. The biggest advantage of this method is that soil strata can be inspected in their natural condition and samples (disturbed or undisturbed) can be conveniently taken. A typical trial pit is shown in Fig. 2.11.

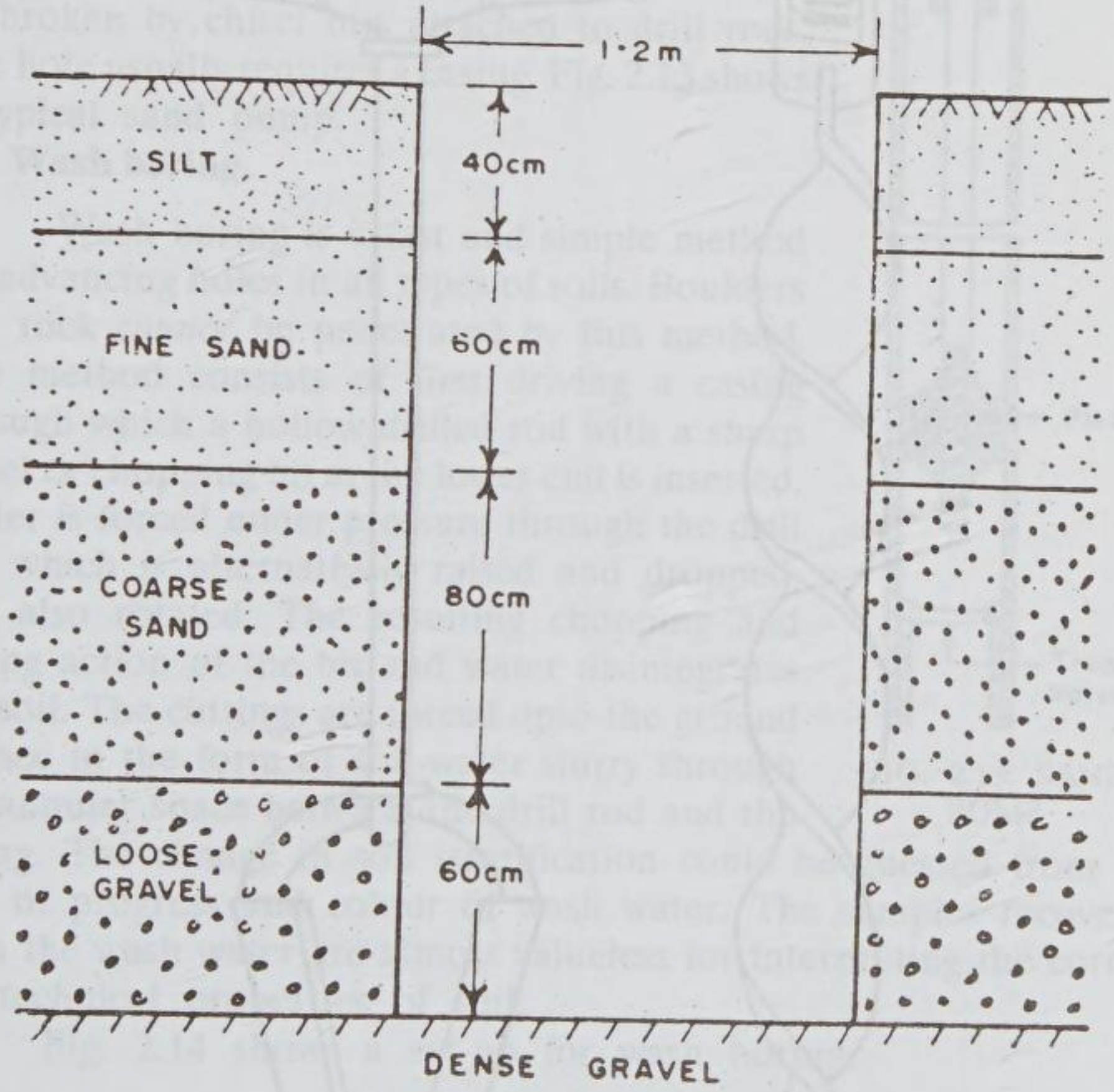


FIG. 2.11. TRIAL PIT

The method is generally considered suitable for shallow depths, say upto 3 m. The cost of open excavation increases rapidly with depth. For greater depths and for excavation below ground water table, specially in pervious soils, measures for lateral support and ground water lowering becomes necessary.

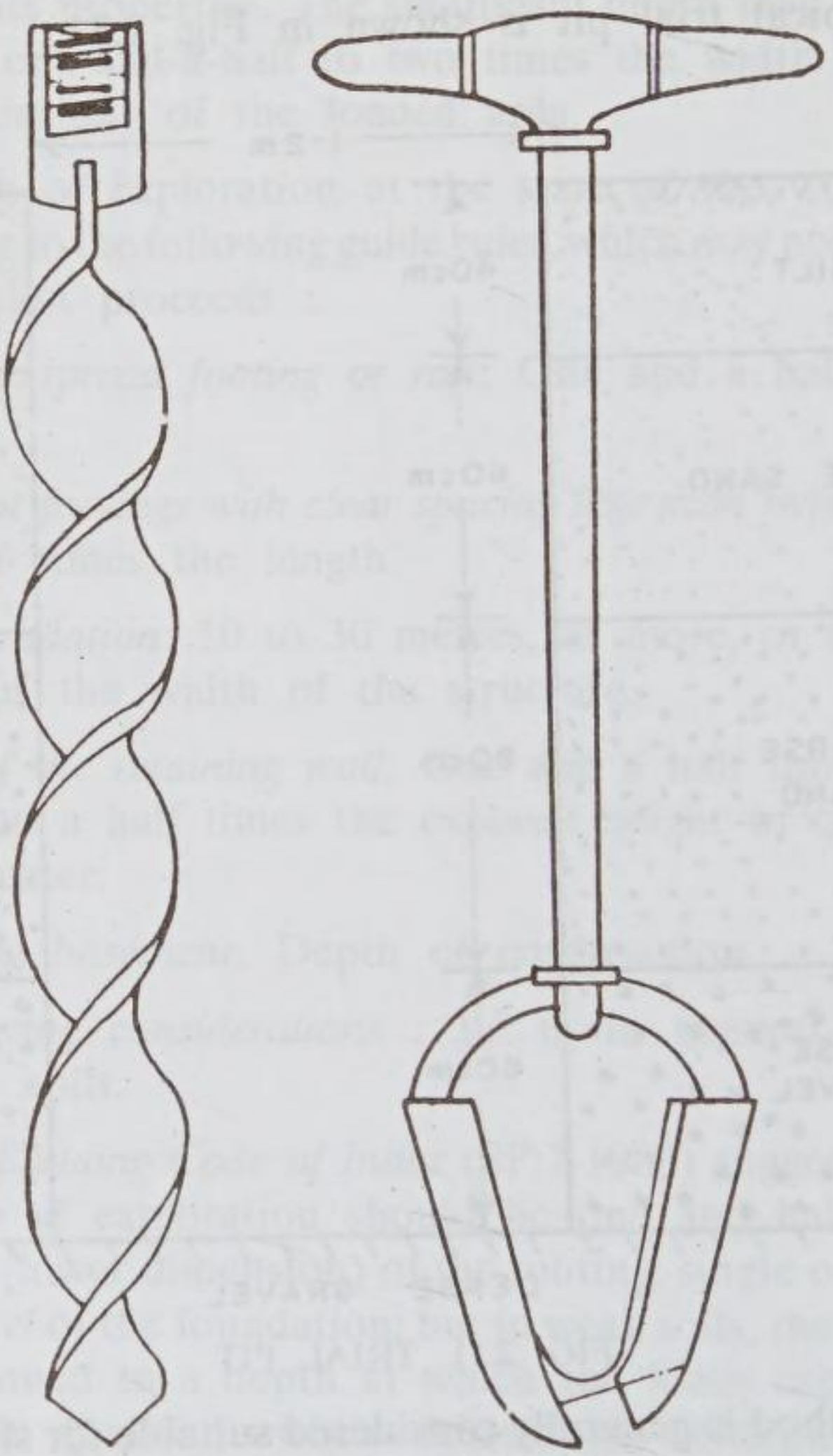
2. BORING METHODS

The following are the various boring methods commonly used:

(*i*) Auger boring (*ii*) Auger and shell boring
(*iii*) Wash boring (*iv*) Percussion boring
and (*v*) Rotary boring.

(*i*) Auger boring

Augers are used in cohesive and other soft soils above water table. They may either be operated manually or mechanically. Hand augers are used upto a depth upto 6 m. Mechanically operated augers are used for greater depths and they can also be used in gravelly soils. Augers are of two types: (*a*) spiral auger and (*b*) post-hole auger.



(a) HELICAL AUGER (*b*) POST-HOLE AUGER.

FIG. 2.12. AUGER.

Samples recovered from the soil brought up by the augers are badly disturbed and are useful for identification purposes only. Auger boring is fairly satisfactory for explorations at shallow depths and for exploratory borrow pits.

(*ii*) Auger and shell boring

Cylindrical augers and shells with cutting edge or teeth at lower end can be used for making deep borings. Hand operated rigs are used for depths upto 25 m and mechanised rigs up to 50 m. Augers are suitable for soft to stiff clays, shells for very stiff and hard clays, and shells or sand pumps for sandy soils. Small boulders, thin soft strata or rock or cemented gravel can be broken by chisel bits attached to drill rods. The hole usually requires a casing. Fig. 2.13 shows a typical sand pump.

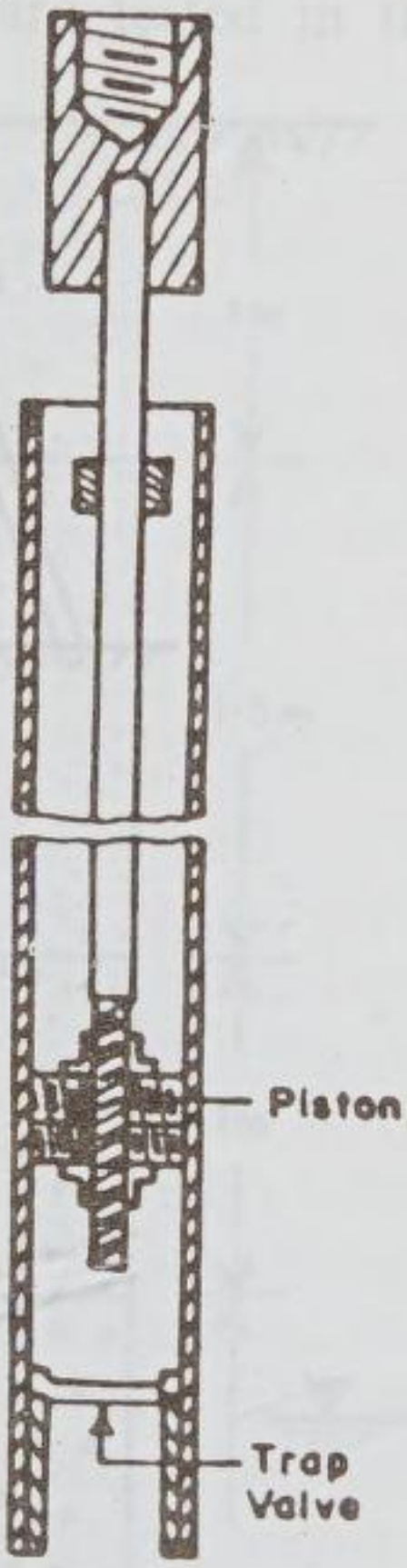


FIG. 2.13. SAND PUMP

(*iii*) Wash boring

Wash boring is a fast and simple method for advancing holes in all types of soils. Boulders and rock *cannot* be penetrated by this method. The method consists of first driving a casing through which a hollow drilled rod with a sharp chisel or chopping bit at the lower end is inserted. Water is forced under pressure through the drill rod which is alternatively raised and dropped, and also rotated. The resulting chopping and jetting action of the bit and water disintegrates the soil. The cuttings are forced upto the ground surface in the form of soil-water slurry through the annular space between the drill rod and the casing. The change in soil stratification could be guessed from the rate of progress and colour of wash water. The samples recovered from the wash water are almost valueless for interpreting the correct geo-technical properties of soil.

Fig. 2.14 shows a set-up for wash boring.

(*iv*) Percussion boring

In this method, soil and rock formations are broken by repeated blows of heavy chiesel or bit suspended by a cable or drill rod. Water is added to the hole during boring, if not already present and the slurry of pulverised material is bailed out at intervals. The method is suitable for advancing a hole in all types of soils, boulders and rock. The formations, however, get disturbed by the impact.

(*v*) Rotary boring

Rotary boring or *rotary drilling* is a very fast method of advancing hole in both rocks and soils. A drill bit, fixed to the lower end of the drill rods, is rotated by a suitable chuck, and is always kept in firm contact with the bottom of the hole. A drilling mud, usually

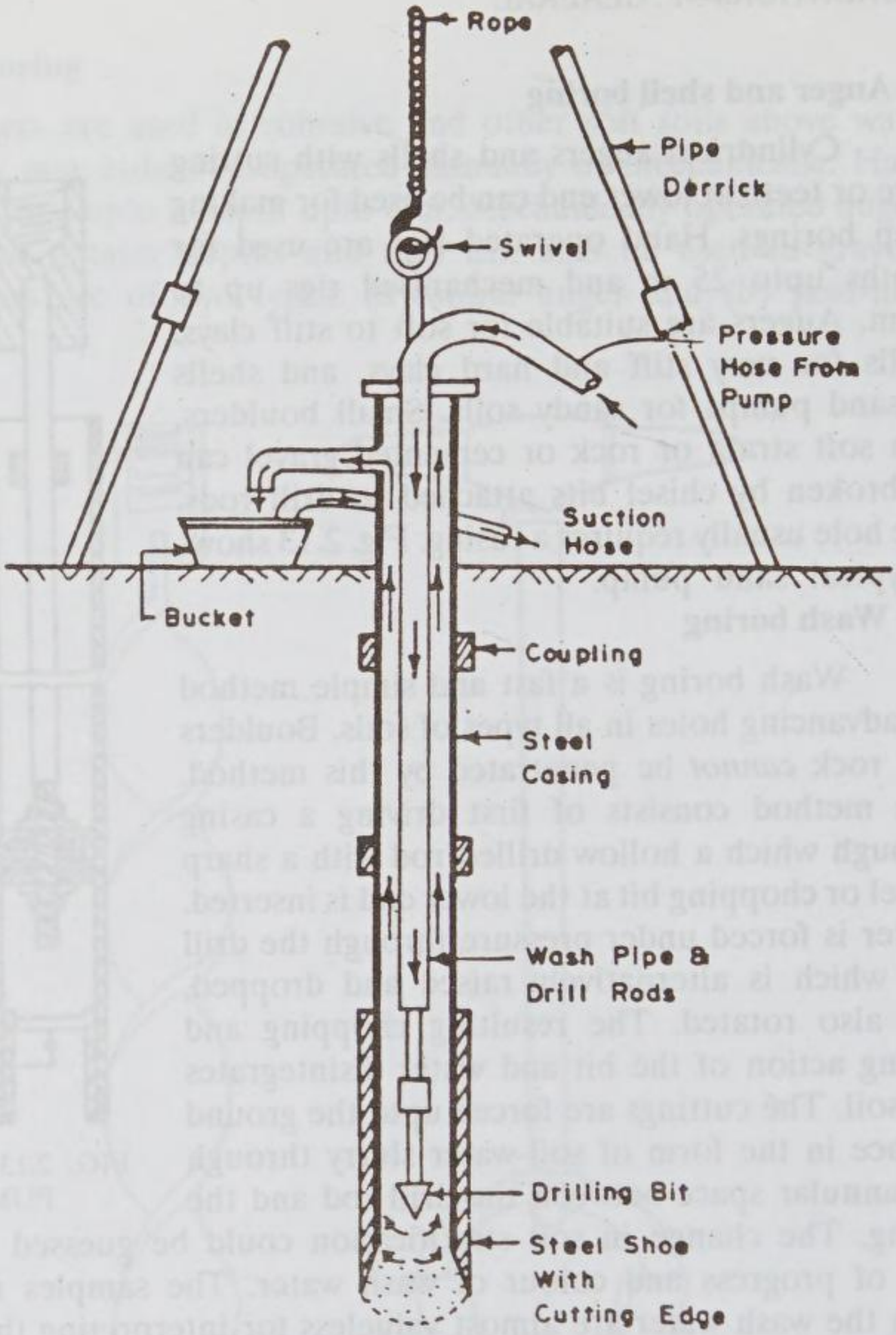


FIG. 2.14. WASH BORING.

a water solution of bentonite, with or without other admixtures, is continuously forced down to the hollow drill rods. The mud returning upwards brings the cuttings to the surface. The method is also known as *mud rotary drilling* and the hole usually requires no casing.

Rotary core barrels, provided with commercial diamond-studded bits or a steel bit with shots, are also used for rotary drilling and simultaneously obtaining the rock cores or samples. The method is then also known as *core boring or core drilling.* Water is circulated down the drill rods during boring.

Record of borings

In all exploration work it is very important to maintain an accurate and explicit record of borings. Soil/rock samples are collected at various depths, during boring. These samples are tested in the laboratory for identification and classification. The samples are suitably preserved and arranged serially according to the depth at which they are found. A boring chart, similar to the one shown in Fig. 2.15 is prepared for each bore hole. A site plan should be prepared, showing the disposition of various bore holes on it.

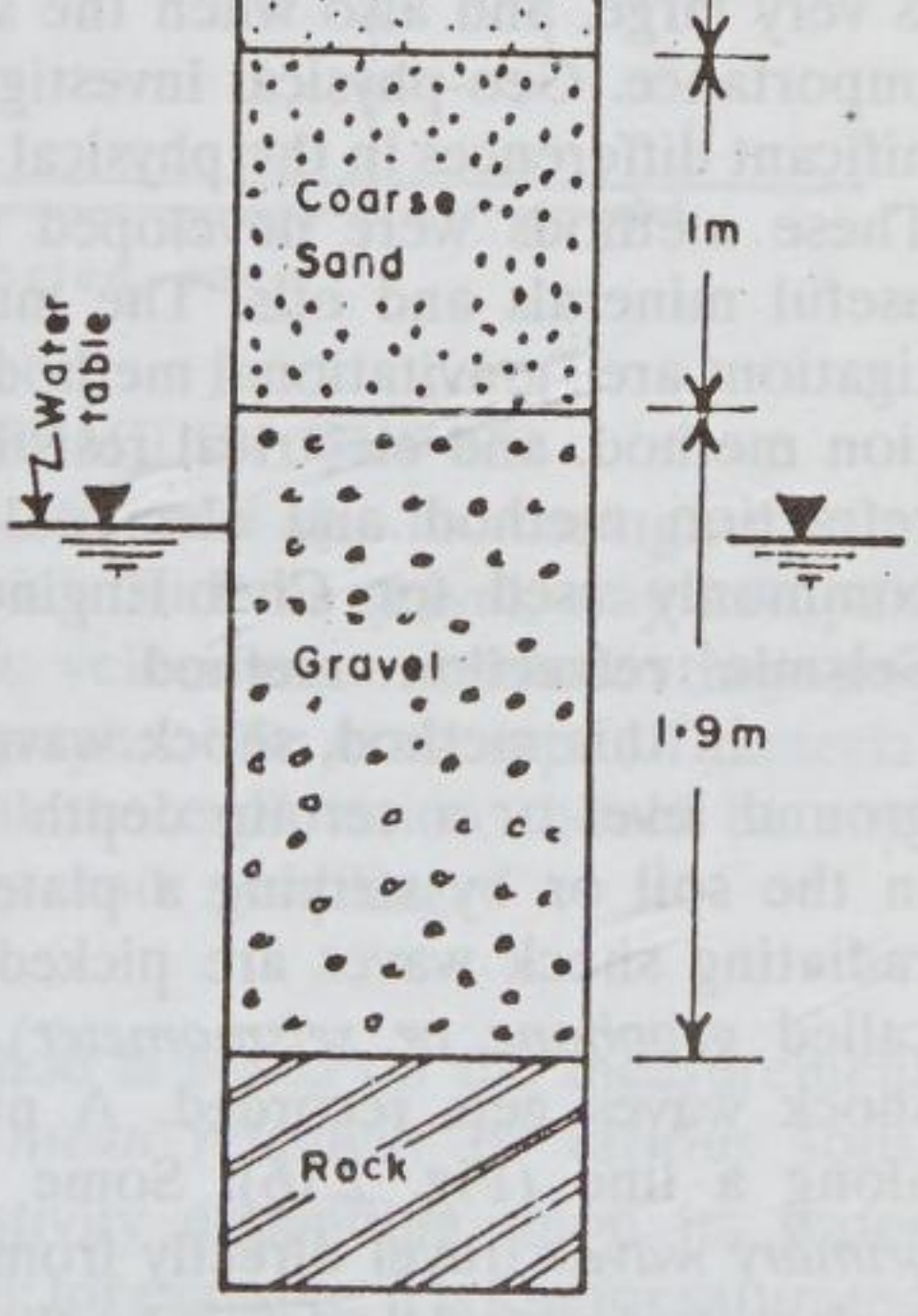


FIG.2.15. DETAILS OF BORING.

Number and disposition of trial pits and borings

The number and disposition of the test pits and borings should be such as to reveal any major changes in the thickness, depth or properties of the strata affected by the works, and the immediate surroundings. The National Building Code of India : (SP : 7–1983) gives the following recommendations for this:

(*a*) for a compact building site covering an area of about 0.4 hectares, one bore hole or trial pit in each corner and one in the centre should be adequate.

(*b*) For small and less important buildings, even one bore hole or trial pit in the centre will suffice.

(*c*) For very large areas covering industrial and residential colonies, the geological nature of the terrain will help in deciding the number of bore holes or trial pits. Dynamic or static cone penetration tests may be performed at every 100 metres by dividing the area into grid patterns and number of bore holes or trial pits decided by examining the variation in the penetration curves.

3. SUB-SURFACE SOUNDINGS

The sounding methods consist of measuring the resistance of the soil with depth by means of *penetrometer* under static or dynamic loading. The penetrometer may consist of a sampling spoon, a cone or other shaped tool. The resistance to penetration is empirically correlated with some of the engineering properties of soil, such as density index, consistency, bearing capacity etc. The value of these tests lie in the amount of experience behind them. These tests are useful for general exploration of erratic soil profiles, for finding depth to bed rock or stratum, and to have an approximate induction of the strength and other properties of soils, particularly for cohesionless soils, from which it is difficult to obtain undisturbed samples. The two commonly used tests are *standard penetration test* and *the cone penetration test.*

4. GEO-PHYSICAL METHODS

Geo-physical methods are used when the depth of exploration is very large, and also when the speed of investigation is of primary importance. Geo-physical investigations involve the detection of significant differences in the physical properties of geological formations. These methods were developed in connection with prospecting of useful minerals and oils. The major method of geo-physical investigations are : gravitational methods, magnetic methods, seismic refraction method, and electrical resistivity method. Out of these, seismic refraction method and electrical resistivity methods are the most commonly used for Civil Engineering purposes.

Seismic refraction method

In this method, shock waves are created into the soil at their ground level or a certain depth below it by *exploding* small charge in the soil or by striking a plate on the soil with a hammer. The radiating shock waves are picked up by the *vibration detector* (also called *geophone or seismometer*) where the time of travel of the shock waves gets recorded. A number of geophones are arranged along a line (Fig. 2.16). Some of the waves, known as *direct or primary waves* travel directly from the shock point along the ground surface and are picked *first* by the geophone. The other waves which travel through the soil get refracted at the interface of two soil strata. The refracted rays are also picked up by the geophone. If the underlying layer is denser, the refracted waves travel much faster. As the distance between the shock point and the geophone increases, the refracted waves are able to reach the geophone earlier than the direct waves. By knowing the time of travel primary and refracted waves at various geophones, the depth of various strata can be evaluated, by preparing distance-time graphs and using analytical methods.

Seismic refraction method is fast and reliable in establishing profiles of different strata provided the deeper layer have increasingly

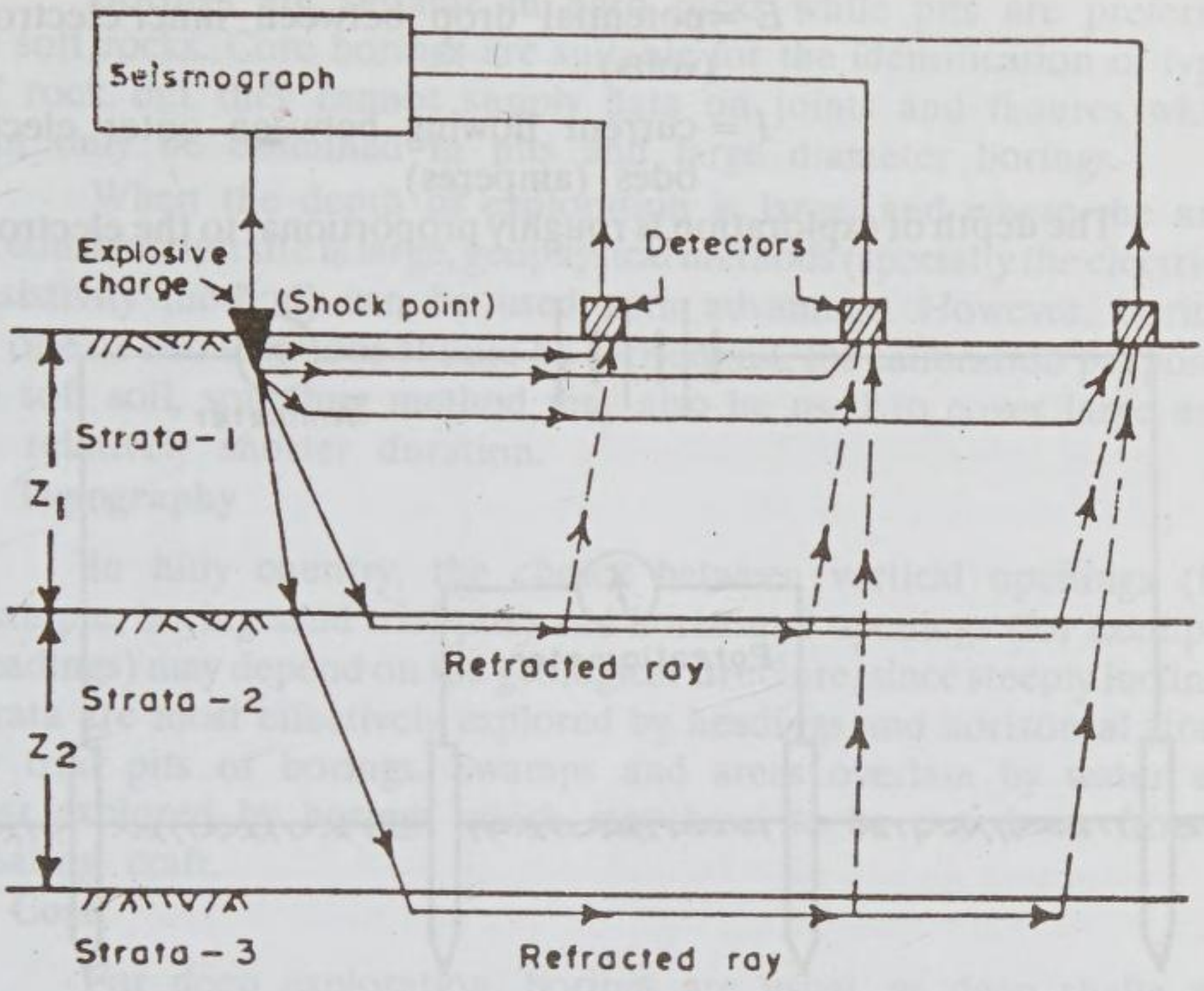


FIG. 2.16. SEISMIC REFRACTION METHOD.

greater density and thus higher velocities and also increasingly greater thickness. Different kinds of materials such as gravel, clay hardpan, or rock have characteristic seismic velocities and hence they may be identified by the distance-time graphs. The exact type of material cannot, however, be recognised and the exploration should be supplemented by boring or soundings and sampling.

Electrical Resistivity Method

The electrical resistivity method is based on the measurement and recording of changes in the *mean resistivity* of various soils.

Each soil has its own resistivity depending upon its water content, compaction and composition; for example, it is low for saturated silt and high for loose dry gravel or solid rock.

The test is conducted by driving four metal spikes to serve as electrodes into the ground along a straight line at equal distance. A direct voltage is imposed between the two outer electrodes, and the potential drop is measured between the inner electrodes. The mean resistivity Ω (ohm-cm) is computed from the expression

$$\Omega = 2\pi D \frac{E}{I} \qquad ...(2.1)$$

where D = distance between the electrodes (cm)

E = potential drop between inner electrodes (volts)

I = current flowing between outer electrodes (amperes)

The depth of exploration is roughly proportional to the electrode

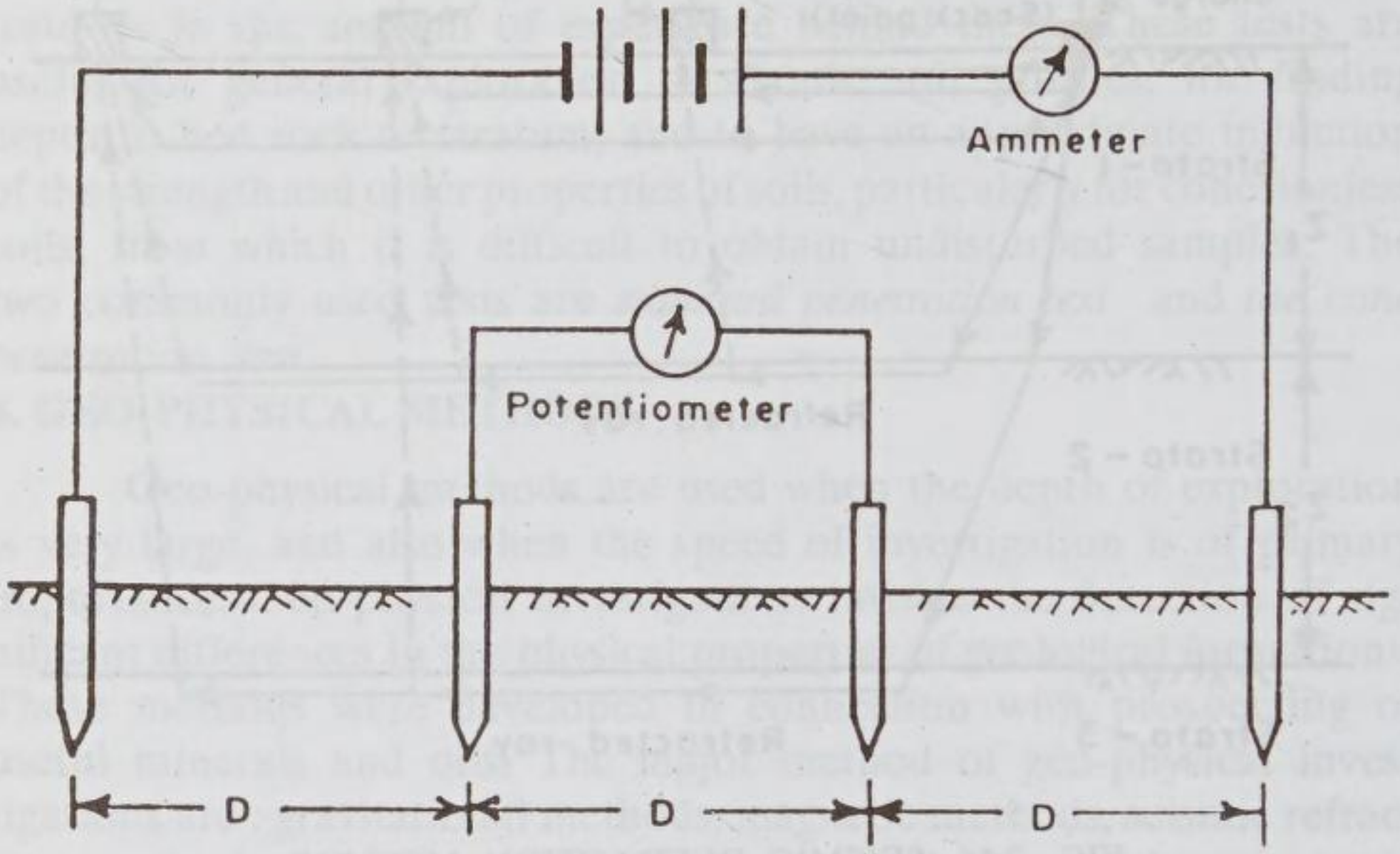


FIG. 2.17. RESISTIVITY METHOD.

spacing. For studying vertical changes in the strata, the electrode system is expanded, about a fixed central point, by increasing the spacing gradually from an initial small value to a distance roughly equal to the depth of exploration required. The method is known as *resistivity sounding.*

To correctly interpret the resistivity data for knowing the nature and distribution of soil formation, it is necessary to make preliminary trial or calibration tests on known formations.

CHOICE OF EXPLORATION METHOD

The choice of a particular exploration method depends on the following factors: (*a*) nature of ground (*b*) topography and (*c*) cost.

1. Nature of ground

In clayey soils, borings are suitable for deep exploration and pits for shallow exploration. In sandy soils, boring is easy but special equipment should be used for taking representative samples below the water table. Such samples can however, be readily taken in trial pits provided that, where necessary, some form of ground water lowering is used.

Borings are suitable in hard rocks while pits are preferred in soft rocks. Core borings are suitable for the identification of types of rock but they cannot supply data on joints and fissures which can only be examined in pits and large diameter borings.

When the depth of exploration is large, and where the area of construction site is large, geophysical methods (specially the electrical resistivity method) can be used with advantage. However, borings at one or two locations should be carried out, for calibration purposes. In soft soil, sounding method may also be used to cover large area in relatively shorter duration.

2. Topography

In hilly country, the choice between vertical openings (for example, boring sand trial pits) and horizontal openings (for example, headings) may depend on the geological structure, since steeply inclined strata are most effectively explored by headings and horizontal strata by trial pits or borings. Swamps and areas overlain by water are best explored by borings which may have to be put down from a floating craft.

3. Cost.

For deep exploration, borings are usual, as deep shafts are costly. However, if the area is vast, geophysical methods or sounding methods may be used in conjunction with borings. For shallow exploration in soil, the choice between pit and borings will depend on the nature of the ground and the information required for shallow exploration in rock; the cost of boring a core drill to the site will only be justified if several holes are required; otherwise trial pits will be more economical.

SOIL SAMPLES AND SAMPLERS

Soil samples can be of two types:

(*i*) Disturbed samples.

(*ii*) Undisturbed samples.

A *disturbed sample* is that in which the natural structure of soil gets partly or fully modified and destroyed although with suitable precautions the natural water content may be preserved. Such a soil sample should, however, be *representative* of the natural soil by maintaining the original proportion of the various particles intact. An *undisturbed sample* is that in which the natural structure and properties remain preserved.

The *sample disturbance* depends upon the design of the samplers and the method of sampling. To take undisturbed samples from bore holes properly designed sampling tools are required. The sampling tube when forced into the ground should cause as little remoulding and disturbance as possible. The design features of the sampler,

that govern the degree of disturbance are (*i*) cutting edge (*ii*) inside wall friction and (*iii*) non-return valve.

Fig. 2.18 shows a typical cutting edge of a sampler, with the lower end of the sampler, with the lower end of the sampler tube. The following terms are defined with respect to the diameters marked in Fig. 2.18.

Area ratio

$$= \frac{D_2^2 - D_1^2}{D_1} \times 100$$

Inside clearance

$$= \frac{D_3 - D_1}{D_1} \times 100$$

Outside clearance

$$= \frac{D_2 - D_4}{D_4} \times 100$$

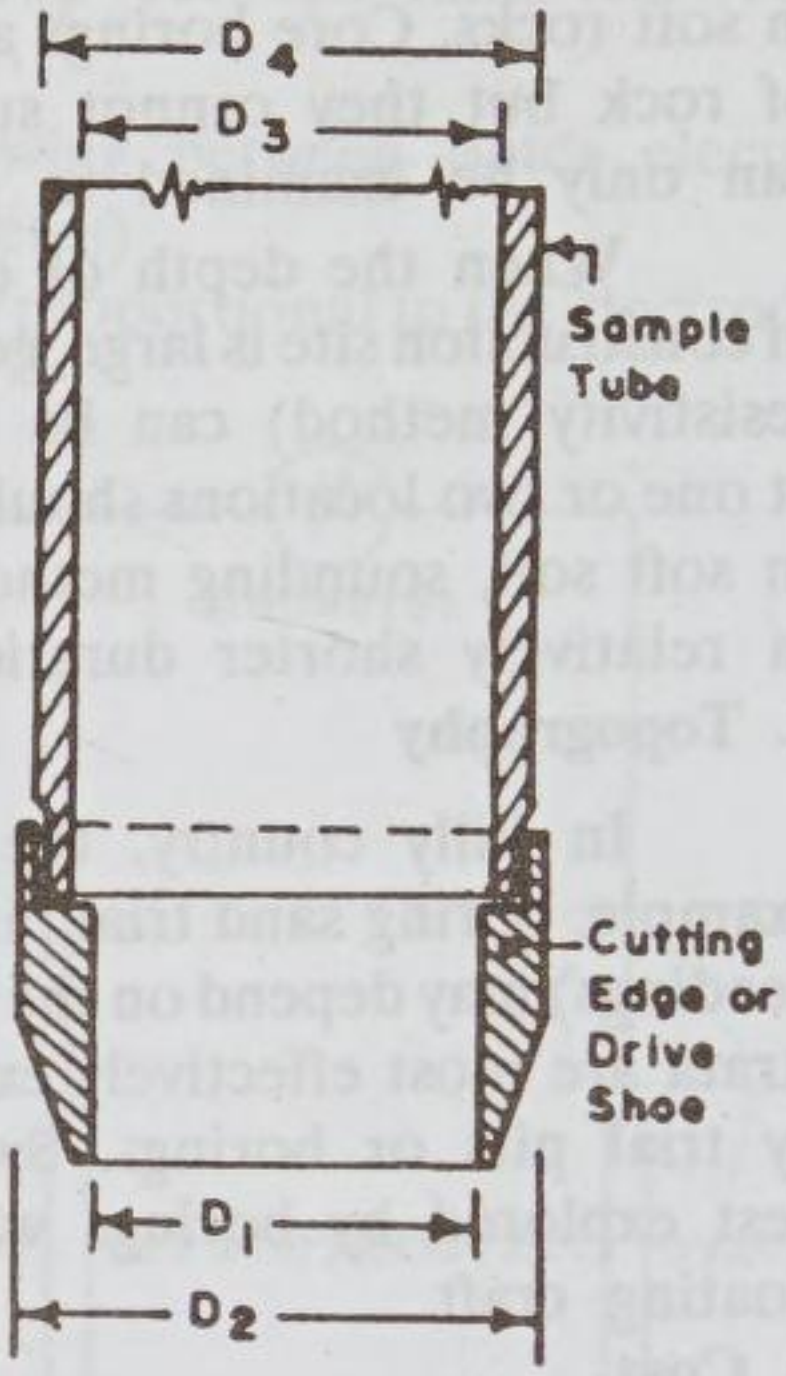


FIG. 2.18. LOWER END OF A SAMPLER.

The area ratio should be as low as possible. It should not be greater than 25 percent; for soft sensitive soil, it should preferably not exceed 10 percent. The inside clearance should lie between 1 to 3 percent and the outside clearance should not be much greater than the inside clearance. The walls of the sampler should be smooth and should be kept properly oiled so that *wall friction* is minimum. Lower value of inside clearance allows the elastic expansion of soil and reduces the frictional drag. The *non-return valve*, invariably provided in samplers, should permit easy and quick escape of water and air when driving the sampler.

Types of Samplers

The samplers are classified as *thick wall or thin wall* samplers depending upon the area ratio. Thick wall samplers are those having the area ratio greater than 10 percent. Depending upon the mode of operation, samplers may be classified in the following three common types : (*i*) open drive sampler (including split spoon samplers), (*ii*) stationary piston sampler and (*iii*) rotary sampler.

The *open drive sampler* is a tube open at its lower end. The sampler head is provided with vents (valve) to permit water and air to escape during driving. The check valve helps to retain sample when the sampler is lifted up. The tube may be seamless or it may be split in two parts; in the latter case it is known as *split spoon sampler*.

The *stationary piston sampler* consists of a sample cylinder and the piston system. During lowering of the sampler through the hole, the lower end of the sampler is kept closed with the piston. When the desired sampling elevation is reached, the piston rod is clamped, thereby keeping the piston stationary, and the sampler tube is advanced down into the soil. The sampler is then lifted up, with piston rod clamped in position. The sampler is more suitable for sampling soft soils saturated sands.

Rotary samplers are the core barrel type having an outer tube provided with cutting teeth and a removable thin wall liner inside. It is used for firm to hard cohesive soils and cemented soils.

2.7. BEARING CAPACITY OF SOILS

As stated earlier, a foundation should be designed to satisfy two essential conditions:

(*i*) It must have some specified safety against ultimate failure.

(*ii*) The settlements under working loads should not exceed the allowable limits for the super-structure.

The *bearing capacity* of the soil, used for the design of foundations (*i.e.* for determining the dimensions of the foundations) is determined on the basis of the above two criteria.

In general, the supporting power of a soil or rock is referred to as its *bearing capacity*. The term bearing capacity is defined after attaching certain qualifying prefixes, as defined below:

1. *Gross pressure intensity* (q). The gross pressure intensity q is the total pressure at the base of the footing due to the weight of the super-structure, self weight of the footing and the weight of the earth fill, if any.

2. *Net Pressure intensity* (q_n). It is defined as the excess pressure, or the difference in intensities of the gross pressure after the construction of the structure and the original overburden pressure. Thus, if D is the depth of the footing

$$q_n = q - \gamma D \qquad ...(2.2)$$

where γ is the unit weight of soil above the level of footing.

3. *Ultimate bearing capacity* (q_f) The ultimate bearing capacity is defined as the minimum gross pressure intensity at the base of the foundation at which the soil fails in shear.

4. *Net ultimate bearing capacity* (q_{nf}). It is the minimum net pressure intensity causing shear failure of the soil. The ultimate bearing capacity q_f and net ultimate bearing capacity (q_{nf}) are evidently connected by the relation

$$q_f = q_{nf} + \gamma D \qquad ...(2.3)$$

5. *Net safe bearing capacity* (q_{ns}). The net safe bearing capacity is the net ultimate bearing capacity divided by a factor of safety F:

$$q_{ns} = \frac{q_{nf}}{F} \qquad ...(2.4)$$

6. *Safe bearing capacity* (q_s). The maximum pressure which the soil can carry safely without risk of shear failure is called the safe bearing capacity. It is equal to the net safe bearing capacity plus original overburden pressure

$$q_s = q_{ns} + \gamma D = \frac{q_{nf}}{F} + \gamma D \qquad ...(2.5)$$

Sometimes, the safe bearing capacity is also referred to as the ultimate bearing capacity q_f divided by a factor of safety F.

7. *Allowable bearing pressure* (q_a). It is the net loading intensity at which neither the soil fails in shear nor there is excessive settlement detrimental to the structure in question. The allowable bearing pressure thus depends both on the sub-soil and the type of building concerned, and is generally less than, and never exceeds, the safe bearing capacity.

Methods of Estimating Bearing capacity

The bearing capacity of soil can be determined by the following methods:

(*a*) Anaiytical methods involving the use of soil parameters
(*b*) Plate load test on the soil
(*c*) Penetration test
(*d*) Presumptive bearing capacity values from codes.

2.8. ANALYTICAL METHODS

A number of analytical methods have been developed to determine the ultimate bearing capacity of soil. These methods use two important shear parameters of soil: (*i*) angle of internal friction ϕ and (*ii*) cohesion c. These parameters are determined in the laboratory, by conducting shear tests on soil samples (preferably, undisturbed samples) collected from the bore holes or test pits. Out of the various theories developed, only two are briefly given here: (*i*) Rankine's analysis and (*ii*) Terzaghi's analysis.

(*a*)- Rankine's Analysis

Rankine considered the equilibrium of two soil elements, one immediately below the foundation (element I) and the other just beyond the edge of the footing (element II), but adjacent to element I. When the load on the footing increases, and approaches a value q_f, *a state of plastic equilibrium* is reached under the footing. For the shear failure of element I, element II must also fail by *lateral thrust* from element I.

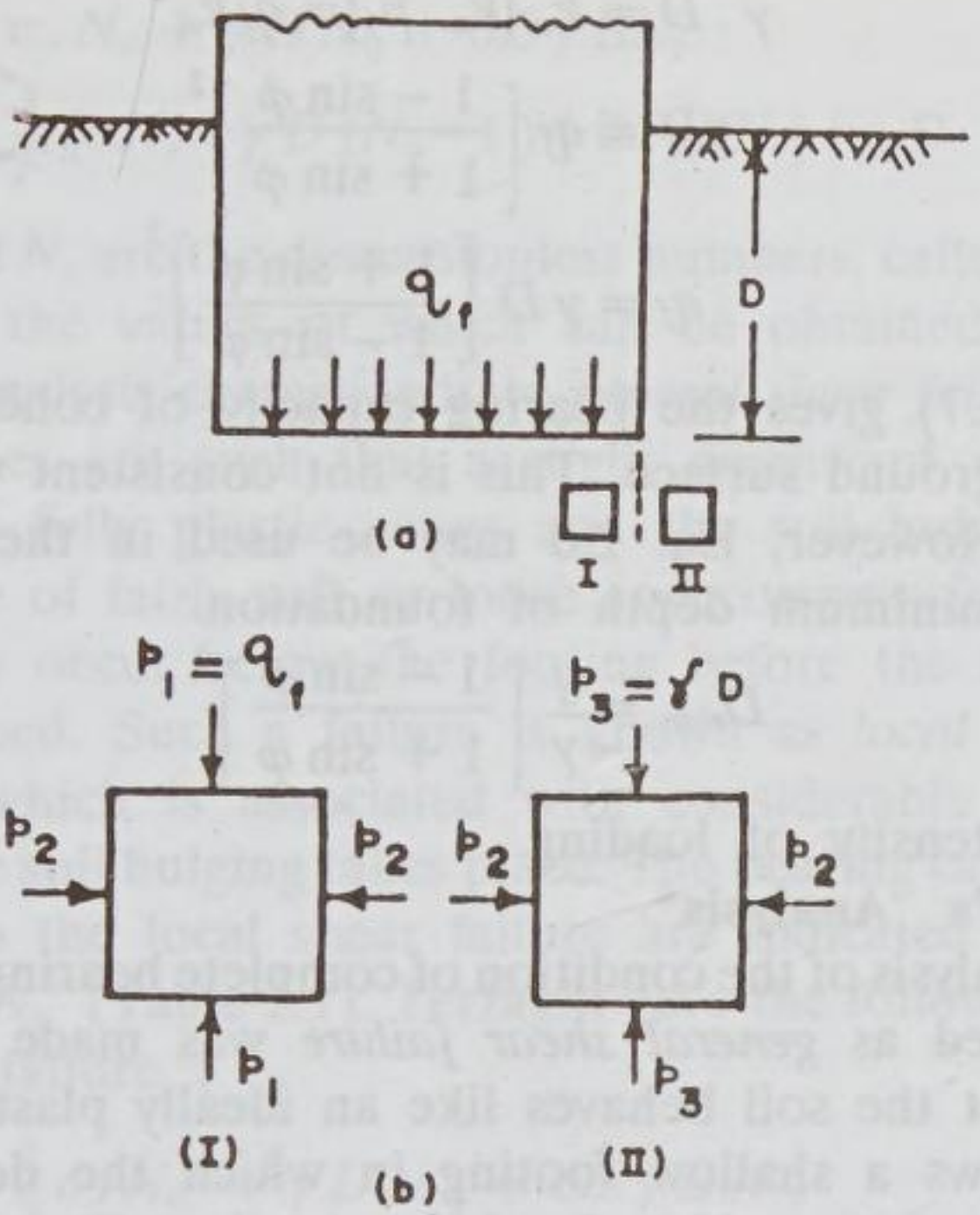


FIG. 2.19

Now, for element I, the major principal stress p_1 from vertical direction is

$$p_1 = q_f$$

According to Rankine's active earth pressure theory the resulting stress p_2 (called the minor stress) in the horizontal direction is given by

$$p_2 = k_a \cdot p_1 \qquad ...(i)$$

(*i.e.* minor principal stress=major principal stress $\times k_a$)
where k_a= *co-efficient of active earth pressure*

$$k_a = \frac{1 - \sin\phi}{1 + \sin\phi},$$

where ϕ is the angle of repose for the soil.

$$\left(\text{If } \phi = 30°,\ k_a = \frac{1 - \sin 30°}{1 + \sin 30°} = \frac{1 - \frac{1}{2}}{1 + \frac{1}{2}} = \frac{1}{3} \right)$$

For element II, the vertical stress p_3 is evidently equal to the weight of overburden $= \gamma D$. However, the stress p_2 in the horizontal direction is the same as found in (*i*) above. However, since p_2 is much more than p_3, major stress on element II is p_2 and minor stress is p_3. From Rankine's earth pressure theory,
minor principal stress= $k_a \times$ (major principal stress)

$$p_3 = k_a \cdot p_2. \qquad ...(ii)$$

Substituting the values of p_2 and p_3, we get

$$\gamma \,.\, D = k_a \,[k_a \,.\, p_1] = q_f k_a{}^2$$

or
$$\gamma D = q_f \left[\frac{1 - \sin\phi}{1 + \sin\phi}\right]^2 \qquad ...(2.6)$$

Hence
$$q_f = \gamma D \left[\frac{1 + \sin\phi}{1 - \sin\phi}\right]^2 \qquad ...(2.7)$$

Eq. (2.7) gives the bearing capacity of cohesionless soils as zero at the ground surface. This is not consistent with the general experience. However, Eq. 2.6 may be used in the following form to get the minimum depth of foundation.:

$$D_{mtn} = \frac{q}{\gamma}\left[\frac{1 - \sin\phi}{1 + \sin\phi}\right]^2 \qquad ...(2.8)$$

where q= intensity of loading.

(*b*) **Terzaghi's Analysis***

An analysis of the condition of complete bearing capacity failure, usually termed as *general shear failure* was made by Terzaghi by assuming that the soil behaves like an ideally plastic material. Fig. 2.20 (*a*) shows a shallow footing in which the depth D is equal to or less than the width B of the footing. The loaded soil fails along a composite surface $A\,B\,C\,B_1A_1$.

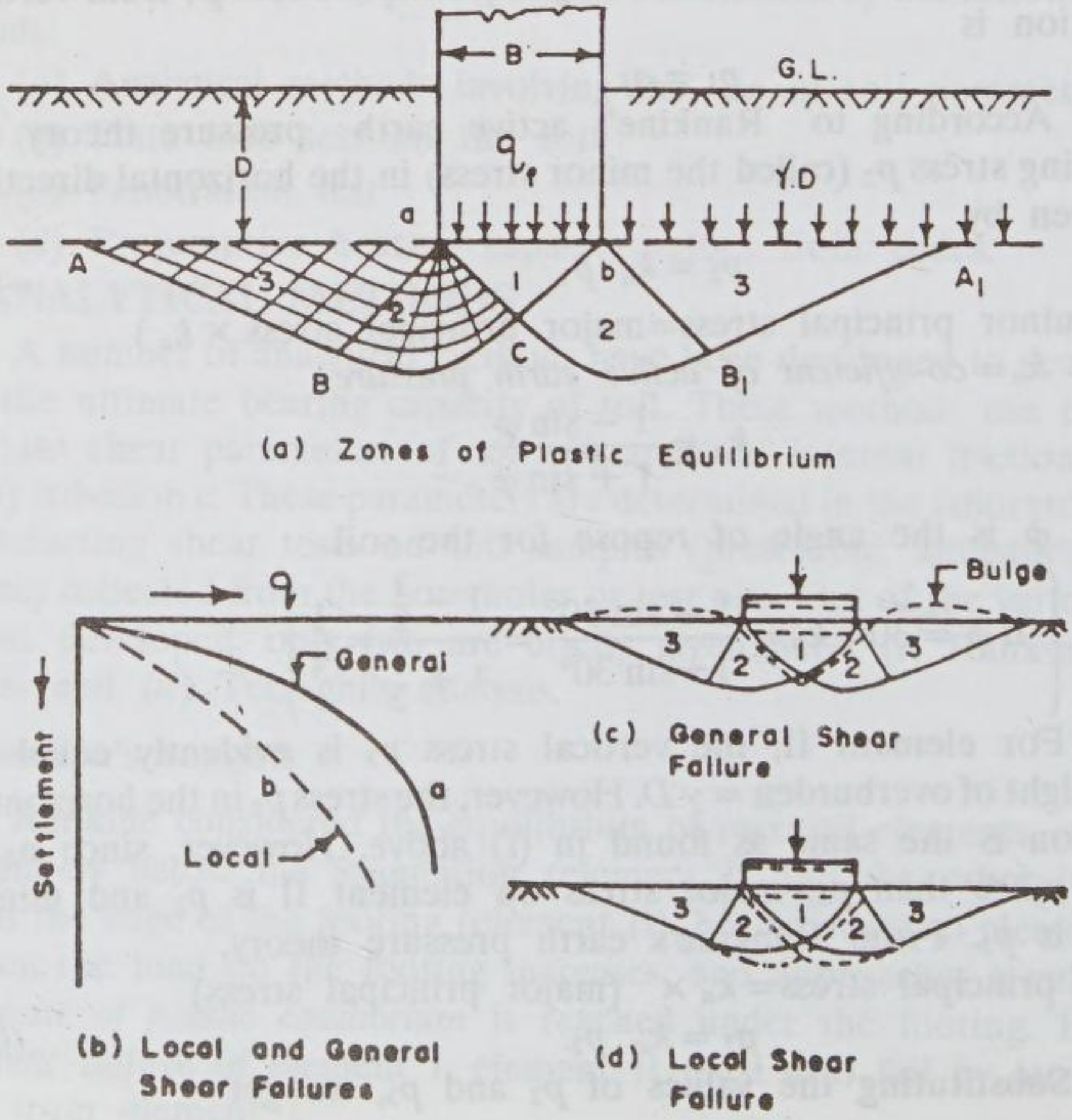


FIG. 2.20. TERZAGHI'S ANALYSIS

Terzaghi gave the following equations:

$$q_f = c\,.\,N_c + \gamma\,D\,N_q + 0.5\,\gamma\,BN_\gamma. \qquad ..(2.9)$$

or $$q_s = \frac{1}{F}[c.N_c + \gamma\,D\,(N_q - 1\,) + 0.5\gamma BN_\gamma] + \gamma\,D. \qquad ..(2.10)$$

where N_c , N_q and N_γ are the dimensionless numbers, called the *bearing capacity factors*, the values of which can be obtained from Table 2.1. The above analysis corresponds to *general shear failure* in which the soil properties are such that a slight downward movement of footing develops fully plastic zones and the soil bulges out [Fig. 2.20 (*c*)]. In case of fairly soft or loose and compressible soil, large deformation may occur below the footing before the failure zones are fully developed. Such a failure is known as *local shear failure* [Fig. 2.20 (*d*)] which is associated with considerable vertical soil movement before soil bulging takes place. The bearing capacity factors corresponding to the local shear failure are indicated with dashes, *i.e.* N_c' , N_q' and N_γ' (Table 2.1). Terzaghi gave the following equation for local shear failure :

$$q_f = \frac{2}{3}c\,.\,N_c' + \gamma\,D\,N'_q + 0.5\,\gamma\,B\,N'_\gamma \qquad ...(2.11)$$

or $$q_s = \frac{1}{F}[\,\frac{2}{3}c\,.\,N_c' + \gamma D(N_q' - 1) + 0.5\,\gamma\,BN_\gamma'] + \gamma\,D \qquad ...(2.12)$$

TABLE 2.1 TERZAGHI'S BEARING CAPACITY FACTORS

ϕ	*General shear failure*			*Local shear failure*		
	N_c	N_q	N_γ	N_c'	N_q'	N_γ'
0	5.7	1.0	0.0	5.7	1.0	0.0
5	7.3	1.6	0.5	6.7	1.4	0.2
10	9.6	2.7	1.2	8.0	1.9	0.5
15	12.9	4.4	2.5	9.7	2.7	0.9
20	17.7	7.4	5.0	11.8	3.9	1.7
25	25.1	12.7	9.7	14.8	5.6	3.2
30	37.2	22.5	19.7	19.0	8.3	5.7
34	52.6	36.5	35.0	23.7	11.7	9.0
35	57.8	41.4	42.4	25.2	12.6	10.1
40	95.7	81.3	100.4	34.9	20.5	18.8
45	172.3	173.3	297.5	51.2	35.1	37.7
48	258.3	287.9	780.1	66.8	50.5	60.4
50	347.5	415.1	1153.2	81.3	65.6	87.1

2.9. PLATE LOAD TEST

Plate Load Test is a field test to determine the ultimate bearing capacity of soil, and the probable settlement under a given loading. The test essentially consists in loading a rigid plate (usually of steel) at the foundation level, and determining the settlements corresponding to each load increment. The ultimate bearing capacity is then taken as the load at which the plate starts sinking at a rapid rate. The

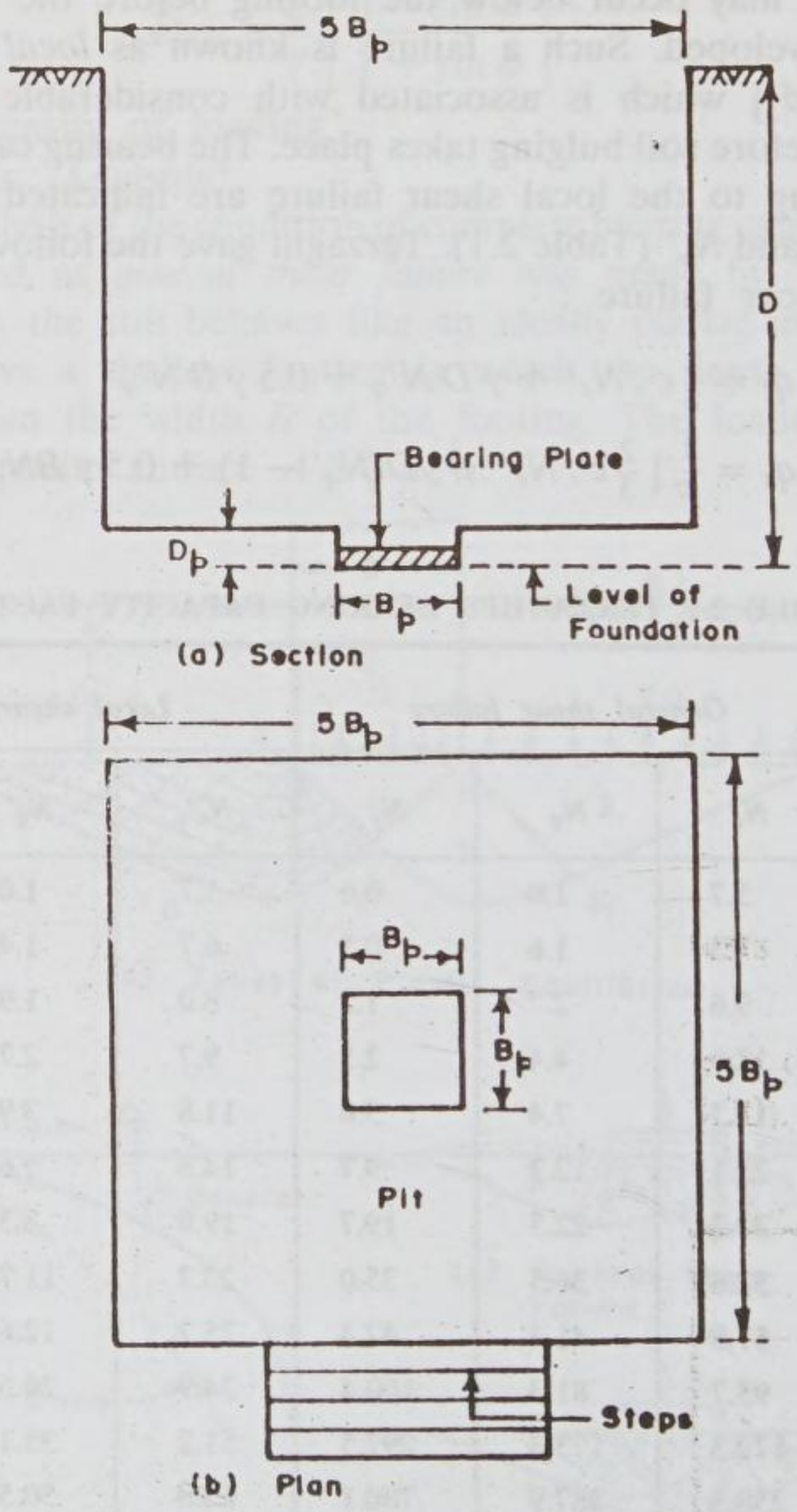


FIG. 2.21. TEST PIT.

method assumes that down to the depth of influence of stresses, the soil strata is reasonably uniform.

The bearing plate is square, of minimum recommended size 30 cm square, and maximum size, 75 cm square. The plate is machined on sides and edges, and should have a thickness sufficient to withstand effectively any bending stresses that would be caused by the maximum

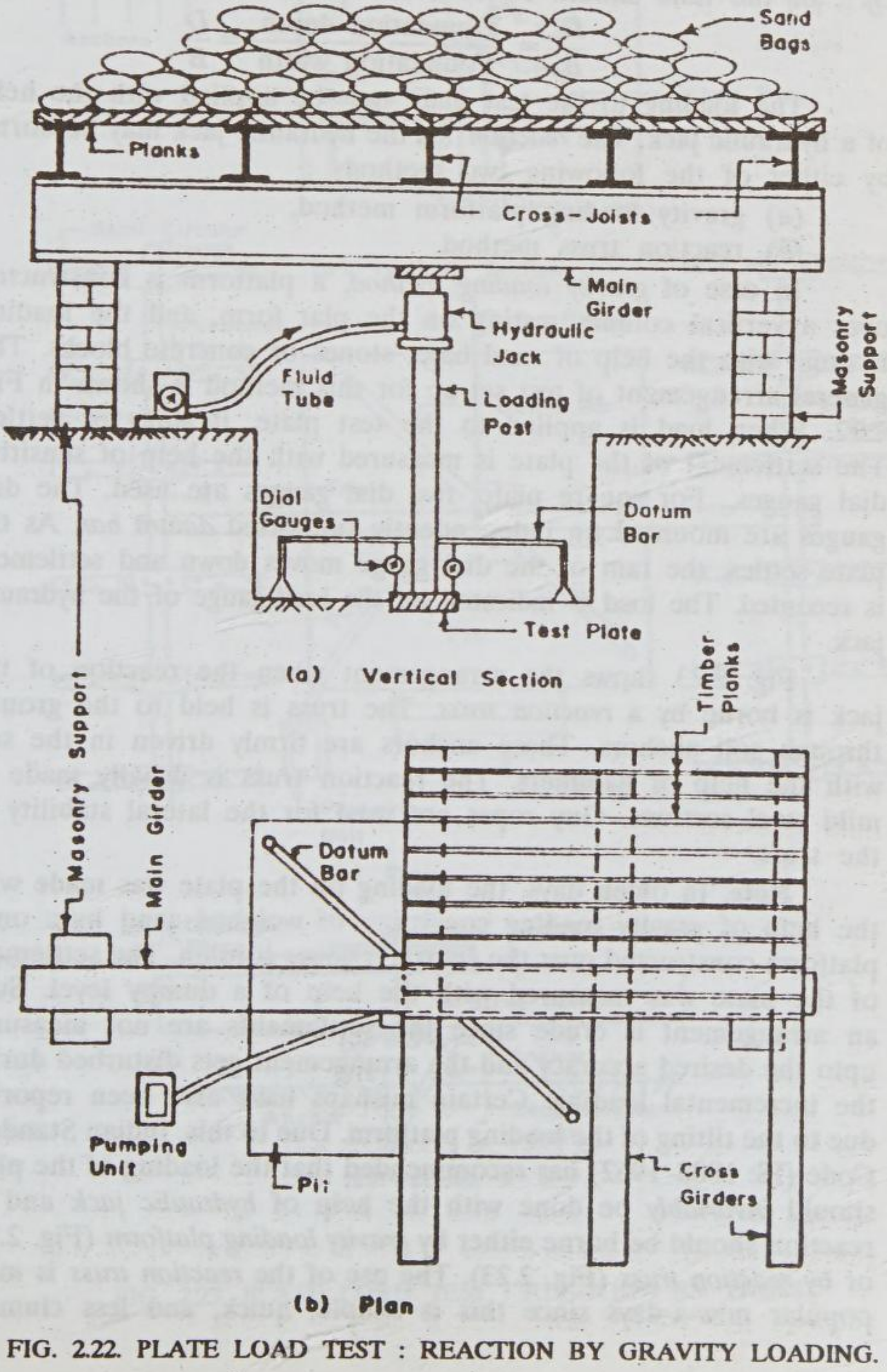


FIG. 2.22. PLATE LOAD TEST : REACTION BY GRAVITY LOADING.

anticipated load. The thickness of steel plate should not be less than 25 mm.

The test pit width is made five times the width of plate (B_p). At the centre of the pit, a small square hole is dug whose size is equal to the size of the plate and the bottom level of which corresponds to the level of actual foundation (Fig. 2.21). The depth D_p , of the hole should be such that

$$\frac{D_p}{B_p} = \frac{\text{Foundation depth}}{\text{Foundation width}} = \frac{D}{B}$$

The loading to the test plate may be applied with the help of a hydraulic jack. The *reaction* of the hydraulic jack may be borne by either of the following two methods :

(*a*) gravity loading platform method,
(*b*) reaction truss method.

In case of *gravity loading method,* a platform is constructed over a vertical column resting on the plat form, and the loading is done with the help of sand bags, stones or concrete blocks. The general arrangement of test set-up for this method is shown in Fig. 2.22. When load is applied to the test plate, it sinks or settles. The settlement of the plate is measured with the help of sensitive dial gauges,. For square plate, two dial gauges are used. The dial gauges are mounted on independently supported *datum bar.* As the plate settles, the ram of the dial gauge moves down and settlement is recorded. The load is indicated on the load-gauge of the hydraulic jack.

Fig. 2.23 shows the arrangement when the reaction of the jack is borne by a *reaction truss.* The truss is held to the ground through soil anchors. These anchors are firmly driven in the soil with the help of hammers. The reaction truss is usually made of mild steel sections. Guy ropes are used for the lateral stability of the truss.

Note. In olden days, the loading on the plate was made with the help of gravity loading consisting of weighed sand bags on a platform constructed over the central loading column. The settlement of the plate was measured with the help of a dumpy level. Such an arrangement is crude since the settlements are not measured upto the desired accuracy and the arrangement gets disturbed during the incremental loading. Certain mishaps have also been reported due to the tilting of the loading platform. Due to this, Indian Standard Code (IS: 1888–1962) has recommended that the loading of the plate should *invariably* be done with the help of *hydraulic jack* and its reaction should be borne either by *gravity loading platform* (Fig. 2.22) or by *reaction truss* (Fig. 2.23). The use of the *reaction truss* is more popular now-a-days since this is simple, quick, and less clumsy.

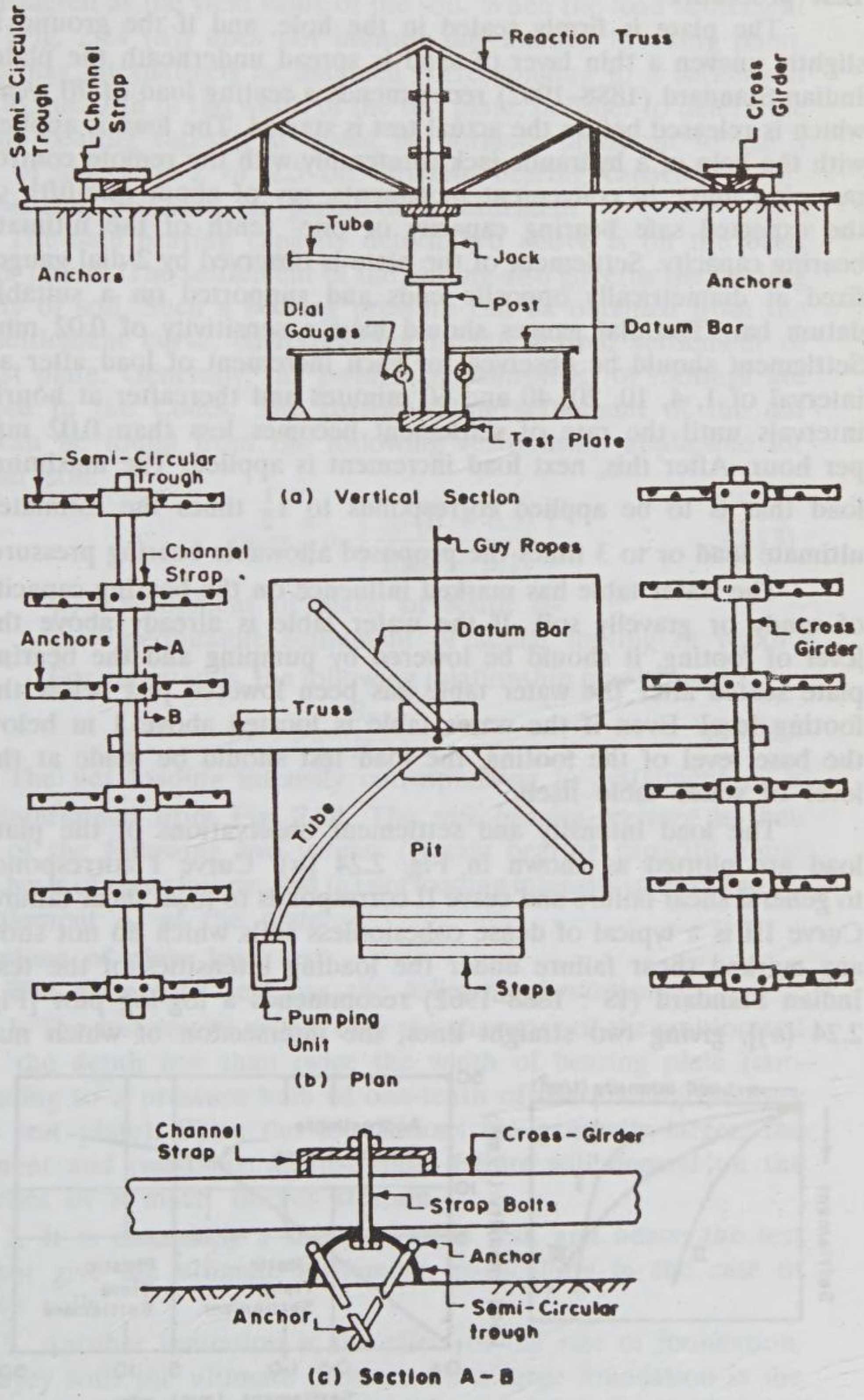


FIG. 2.23. PLATE LOAD TEST : REACTION BY TRUSS.

Test procedure

The plate is firmly seated in the hole, and if the ground is slightly uneven a thin layer of sand is spread underneath the plate. Indian Standard (1888–1962) recommends a seating load of 70 g/cm^2 which is released before the actual test is started. The load is applied with the help of a hydraulic jack (preferably with the remote control pumping unit), in convenient increments, say of about one-fifth of the expected safe bearing capacity or one- tenth of the ultimate bearing capacity. Settlement of the plate is observed by 2 dial gauges fixed at diametrically opposite ends and supported on a suitable datum bar. The dial gauges should have a sensitivity of 0.02 mm. Settlement should be observed for each increment of load after an interval of 1, 4, 10, 20, 40 and 60 minutes and thereafter at hourly intervals until the rate of settlement becomes less than 0.02 mm per hour. After this, next load increment is applied. The maximum load that is to be applied corresponds to $1\frac{1}{2}$ times the estimated ultimate load or to 3 times the proposed allowable bearing pressure.

The water table has marked influence on the bearing capacity of sandy or gravelly soil. If the water table is already above the level of footing, it should be lowered by pumping and the bearing plate seated after the water table has been lowered just below the footing level. Even if the water table is located above 1 m below the base level of the footing, the load test should be made at the level of water table itself.

The load intensity and settlement observations of the plate load are plotted as shown in Fig. 2.24 (*a*). Curve I corresponds to general shear failure and curve II corresponds to local shear failure. Curve III is a typical of dense cohesionless soils which do not show any marked shear failure under the loading intensities of the test. Indian Standard (IS : 1888–1962) recommends a log-log plot [Fig. 2.24 (*b*)], giving two straight lines, the intersection of which may

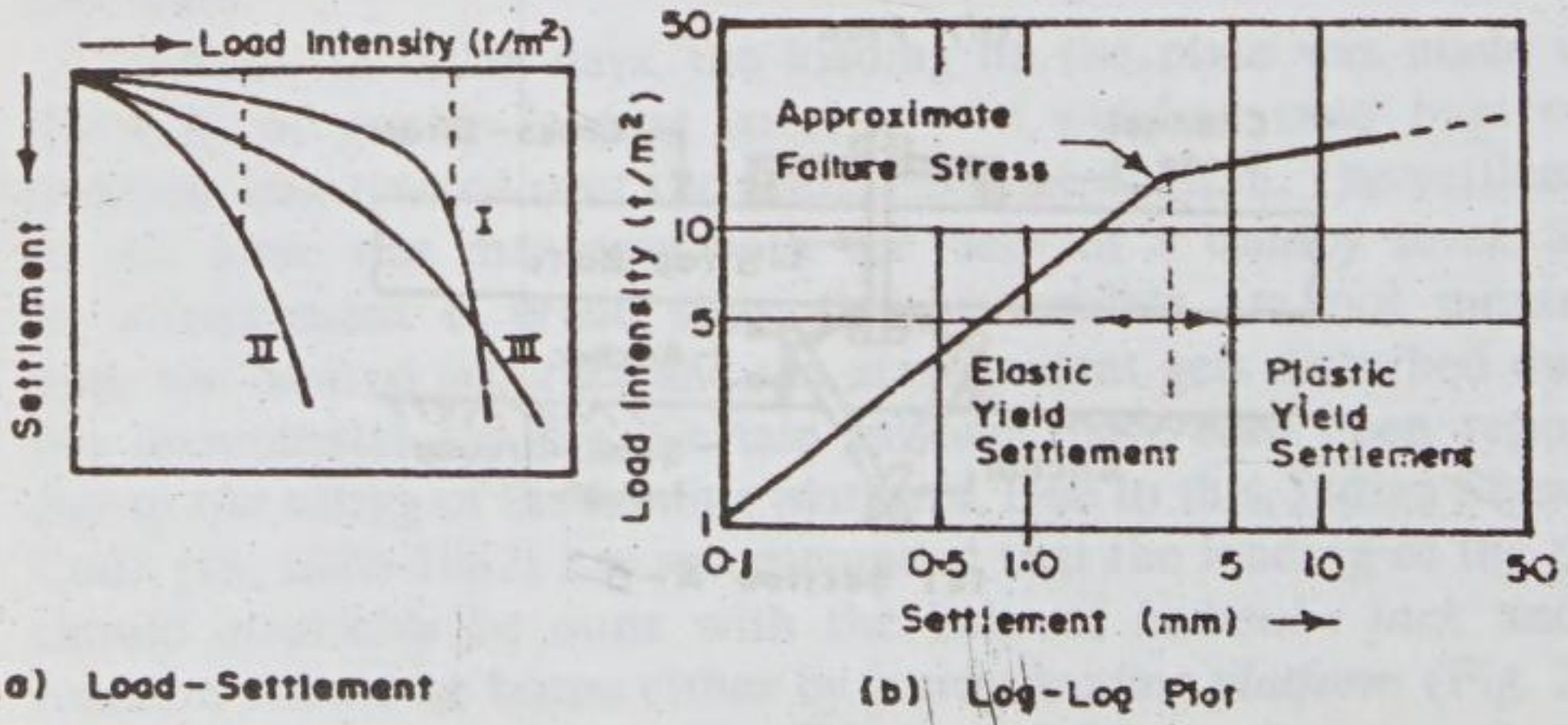


FIG. 2.24. LOAD SETTLEMENT CURVES.

be considered as the yield value of the soil. When the load settlement curve [Fig. 2.24 (*a*)] does not indicate any marked breaking point, failure may alternatively be assumed corresponding to a settlement equal to one-fifth of the width of the test plate. In order to determine the safe bearing capacity it would be normally sufficient to use a factor of safety of 2 or 2.5 on ultimate bearing capacity.

Safe bearing pressure on permissible settlement

The safe bearing capacity determined above is on the basis of *shear failure.* The settlement of the footing also governs the bearing capacity of soil. Such a bearing pressure can be obtained from the load settlement curve, corresponding to the desired settlement of the test plate. Generally, the *permissible settlements* of footings are specified in the Codes. The corresponding settlement of the *test plate* can be found from the following relationship applicable for granular soil:

$$\rho_P = \frac{4}{3}\rho_F \left[\frac{B_P\,(B + 0.3)}{B(B_P + 0.3)}\right]^2 \qquad ...(2.13)$$

where ρ_P = settlement of plate, of width B_P

ρ_F = permissible settlement of actual footing of width B

For clayey soils, the following relationship may be used :

$$\rho_P = \rho_E \cdot \frac{B_P}{B} \qquad ...(2.14)$$

The net loading intensity corresponding to settlement ρ_P is then determined from Fig. 2.24. The *safe bearing pressure* is then lesser of the following two values: (*i*) safe bearing capacity found on the basis of shear failure, and (*ii*) net loading intensity corresponding to settlement ρ_P of the plate.

Limitations of plate load test

The plate load test has the following *limitations* :

1. The test results reflect only the character of the soil located within the depth less than twice the width of bearing plate (corresponding to a pressure bulb of one-tenth of the loading intensity at the test plate). Since the foundations are generally larger, the settlement and resistance against shear failure will depend on the properties of a much thicker stratum.

2. It is essentially a short duration test, and hence the test does not give the ultimate settlement, particularly in the case of cohesive soils.

3. Another limitation is the effect of the size of foundation. For clayey soils the ultimate pressure for a large foundation is the *same* as that for the test plate. But in dense sandy soils, the bearing capacity *increases* with the size of the foundation, and the tests on smaller size bearing plates tend to give conservative values.

2.10. PENETRATION TESTS

These tests involve the measurements of the resistance to penetration of a sampling spoon, a cone or other shaped tool under dynamic or static loadings. The resistance is empirically correlated with some of the engineering properties of soil, such as density index, bearing capacity etc. Two commonly used penetration tests are

(*i*) Standard penetration test,

(*ii*) Dutch cone test.

Standard Penetration Test

The test (IS : 2131–1963) is performed in a clean hole, 55 to 150 mm in diameter. A casing or drilling mud may be used to support the sides of the hole. A thick wall split tube sampler, 50.8 mm outer dia. and 35 mm internal dia. is driven into the undisturbed soil at the bottom of the hole under the blows of 65 kg drive weight with 75 cm free fall. The minimum open length of the sampler should be 60 cm. The sampler is first driven through 15 cm as a seating drive. It is further driven through 30 cm and the number of blows required for this are counted. This number of blows is termed as *penetration resistance N.*

In very fine, or silty, saturated sand, an apparent increase in resistance occurs. Terzaghi and Peck have recommended the use an equivalent penetration resistance N_e , in place of the actually observed value of N, when N is greater than 15. N_e is given by the following relation:

$$N_e = 15 + \frac{1}{2}(N - 15) \qquad ...(2.15)$$

Terzaghi and Peck's empirical charts for determining net bearing pressure q_ρ for footing on sand depend on B and N value, to limit maximum settlement of individual footing to 2.5 cm and differential settlement of 2 cm, assuming that a differential settlement of 2 cm can be tolerated by most of the ordinary structures. The empirical relations are represented by the following equation:

$$q_\rho = 3.5\,(N-3)\left(\frac{B+0.3}{2B}\right)^2 R_{w2}\, R_d \qquad ...(2.16)$$

where q_ρ = allowable *net* increase in soil pressure over exising soil pressure for settlement of 2.5 cm, in t/m^2

N = standard penetration number, with applicable corrections

B = width of footing (or least lateral dimension), in metres

R_{w2} = water reduction factor $= 0.5\left(1 + \frac{Z_{w2}}{B}\right) \le 1$

Z_{w1} = depth of water table *below* the level of footing. If the water table is above the base of footing, R_{w2}

should be taken as 0.5

$$R_d = \text{depth factor} = \left(1 + \frac{0.2\,D}{B}\right) \leq 1.20$$

The standard penetration test is very useful for the design of rafts. The safe bearing value for rafts may be taken as *smaller* of the values of q_1 and q_2 given below:

$$q_1 = 21.4\,N^2B\;R_{w1} + 64(100 + N^2)\,DR_{w2} \quad ...(2.17)$$

and $$q_2 = 1950\;(N - 3)\;R_{w2} \quad ...(2.18)$$

where q_1 and q_2 = allowable soil pressure under raft foundation, in kg/m^2 (using a factor of safety of three)

R_{w1} = water reduction factor

$$= 0.5\left(1 + \frac{Z_{w1}}{D}\right) \leq 1$$

Z_{w1} = depth of water table below ground surface. If the water table is at ground level, $R = 0.5$.

Dutch cone test

This test is used for getting a continuous record of the resistance of soil by penetrating steadily under static pressure a cone with a base of 10 cm^2 (3.6 cm in dia.) and an angle of 60° at vertex. The cone is carried at the lower end of a steel driving rod which passes through a steel tube (mantle) with external diameter equal to the base of the cone. Either the cone or the tube, or both together can be forced into the soil by means of jack. To know the cone resistance, the cone along is first forced down for a distance of 8 cm and the maximum value of resistance is recorded. The steel tube is then pushed down upto the cone, and both together are further penetrated through a depth of 20 cm to give the total of cone resistance and the frictional resistance along the tube.

The cone test is considered very useful in determining the bearing capacity of pits in cohesionless soils, particularly in fine sands of varying density. The cone resistance q_c (kg/cm^2)is approximately equal to 5 to 10 times the penetration resistance N.

2.11. PRESUMPTIVE BEARING CAPACITY VALUES FROM CODES

For the design of foundations of lightly loaded structures and for a preliminary design of any structure the presumptive safe bearing capacity may be used. The presumptive safe bearing capacities of various types of soils are given in Table 2.2.

TABLE 2.2
PRESUMPTIVE SAFE BEARING CAPACITY (IS : 1904-1961)

S.No.	*Type of rocks and soils*	*Presumptive safe bearing capacity*		
		Kg/cm²	*t/m²*	*kN/m²*
	(a) Rocks			
1.	Rocks (hard) without laminations and defects, for example, granite, trap and diorite.	33	330	3300
2.	Laminated rocks, for example, sand stone and lime stone in sound conditions.	16.5	165	1650
3.	Residual deposits of shattered and broken bed rock and hard shale, cemented material .	9.0	90	900
4.	Soft rock.	4.5	45	450
	(b) Non-cohesive soils			
5.	Gravel, sand and gravel, compact and offering high resistance to penetration when excavated by tools (Note 1).	4.5	45	450
6.	Coarse sand, compact and dry (with ground water level at a depth greater than width of foundation, below the base of footing).	4.5	45	450
7.	Medium sand, compact and dry.	2.5	25	250
8.	Fine sand, silt (dry lumps easily pulverised by fingers).	1.5	15	150
9.	Loose gravel or sand gravel mixture ; loose coarse to medium sand, dry (Note 1).	2.5	25	250
10	Fine sand, loose and dry	1.0	10	100
	(c) Cohesive soils			
11.	Soft shale, hard or stiff clay in deep bed, dry (This ground is susceptible to long term consolidation settlement).	4.5	45	450
12.	Medium clay, readily indented with a thumb nail.	2.5	25	250
13.	Moist clay and sand clay mixture which can be indented with strong thumb pressure.	1.5	15	150
14.	Soft clay indented with moderate thumb pressure.	1.0	10	100
15.	Very soft clay which can be penetrated several centimetres with the thumb.	0.5	5	50

TABLE 2.2. (Contd.)

S.No.	*Type of rocks and soils*	*Presumptive safe bearing capacity*		
		Kg/cm²	*t/m²*	*kN/m²*
16.	Black cotton soil or other shrinkable or expansive clay in dry condition (50 percent saturation).	To be determined after investigation (See Note 2)		
	(d) Peat			
17.	Peat.	To be determined after investigation (See Notes 2 and 3)		
	(e) Made up ground			
18.	Fills or made up ground.	To be determined after investigation (See Notes 1 and 4)		

Note 1. Compactness or looseness of non-cohesive materials may be determined by driving a wooden picket of dimension 5 cm × 5 cm × 70 cm with a sharp point. The picket shall be pushed vertically into the soil by full weight of a person and if the penetration of the picket exceeds 20 cm, the loose state shall be assumed to exist.

Note 2. No generalised values for presumptive safe bearing capacities can be given for these types of soils. In such area, adequate site investigation shall be carried out and expert advice shall be sought.

Note 3. Peat may occur in a very soft spongy condition or may be quite firm and compact. While ultimate bearing capacity may be high in the compact cases, very large consolidation settlements occur even under small pressures and the movements continue for decades.

Note 4. The strength of made-up ground depends on the nature of the material, its depth and age, and the method used for consolidating it.

Note 5. The presumptive safe bearing values may be increased by an amount equal to weight of the material (virgin soil) removed from above the bearing level, that is, the base of the foundation.

Note 6. For non-cohesive soils, the presumptive safe bearing values shall be reduced by 50 percent if the water table is above or near the bearing surface of the soil. If the water table is below the bearing surface of the soil at a distance at least equal to the width of the foundation, no such reduction shall apply. For intermediate depths of the water table, proportional reduction of presumptive safe bearing value may be made.

2.12. SETTLEMENT OF FOUNDATIONS

The vertical downward movement of the base of a structure is called settlement and its effect upon the structure depends on its magnitude, its uniformity, the length of the time over which it takes place, and the nature of the structure itself.

Foundation settlement may be caused by some or a combination of the following reasons:

1. *Elastic compression* of the foundation and the underlying soil.

2. *Inelastic (or plastic) compression* of the underlying soils, which is much larger than the elastic compression. The inelastic compression can be predicted by the theory of consolidation.

3. *Ground water lowering.* Repeated lowering and raising of water level in loose granular soil tends to compact the soil and cause settlement of the ground surface. Lowering of water level in fine grained soils cause consolidation settlement. The major settlements in the city of Maxico has been due to ground water lowering, and due to this, the city has been called as the 'sinking city of Maxico'.

4. *Vibrations* due to pile driving, blasting and oscillating machineries may cause settlement in deposits of granular soils.

5. *Seasonal swelling and shrinkage* of expansive clays.

6. *Ground movement on earth slopes,* such as surface erosion, slow creep or landslide.

7. *Other causes* such as adjacent excavation, mining subsidence, underground erosion, etc.

A certain amount of elastic and inelastic settlement of foundations is unavoidable, and it should be taken into account in design. Provided the settlement is uniform over the whole area of the building and is not excessive, it does little damage. If, however, the amount of settlement varies at different points under the building, giving rise to what is known as relative or differential settlement, stresses will be set up in the structure. These may be relived in the case of brick structure, for example, by the setting up of a large number of cracks at the joints, but in more rigid structures, overstressing of some structural members might occur.

It is suggested that the allowable pressure should be selected such that the maximum settlement of any individual foundation is 2.5 cm. It has also been suggested that the *differential settlement* of uniformly loaded continuous foundation and of equally loaded spread foundations of approximately the same size, is unlikely to exceed half the maximum settlement, and that normal structures such as office buildings and flats can satisfactorily withstand differential settlements of about 18 mm between adjacent columns spaced 6 to 8 m apart.

According to National Building Code of India (SP: 7-1983), the differential settlement shall be kept within limits to which the super-structure can accommodate itself without harmful distortion, by suitably designing the foundation. Total settlements shall be so restricted or special arrangements made so that connections to the building, such as drains, are not damaged. For simple spread footings on sands, the allowable bearing pressure should be such that the differential settlement does not exceed 1/300; this condition is generally

satisfied if the total settlement is limited to 50 mm. For simple spread footings on clayey soils, the allowable bearing pressure should be such that the differential settlement does not exceed 1/300 ; this condition is generally satisfied if the total settlement is limited to 75 mm.

The recommendations of American Codes are based upon the simple logic that if the maximum total settlement is kept within a reasonable limit, the differential settlement will be only a fraction (generally about three-quarters of this limit), depending upon the type of structure and pattern of loading. The allowable maximum settlement values are given below:

Type of structure	*Allowable Maximum Settlement (mm)*
Commercial and institutional buildings	25
Industrial buildings	38
Warehouses	50
Special machinery foundations	Often less than 0.5 mm

According to Polshin and Tokar(1957), brick masonry will crack(due to differential settlement) when the unit elongation amounts to 0.0005. Based on this criterion, the *permissible differential settlement* of brick walls is shown in Fig. 2.25, and is as follows

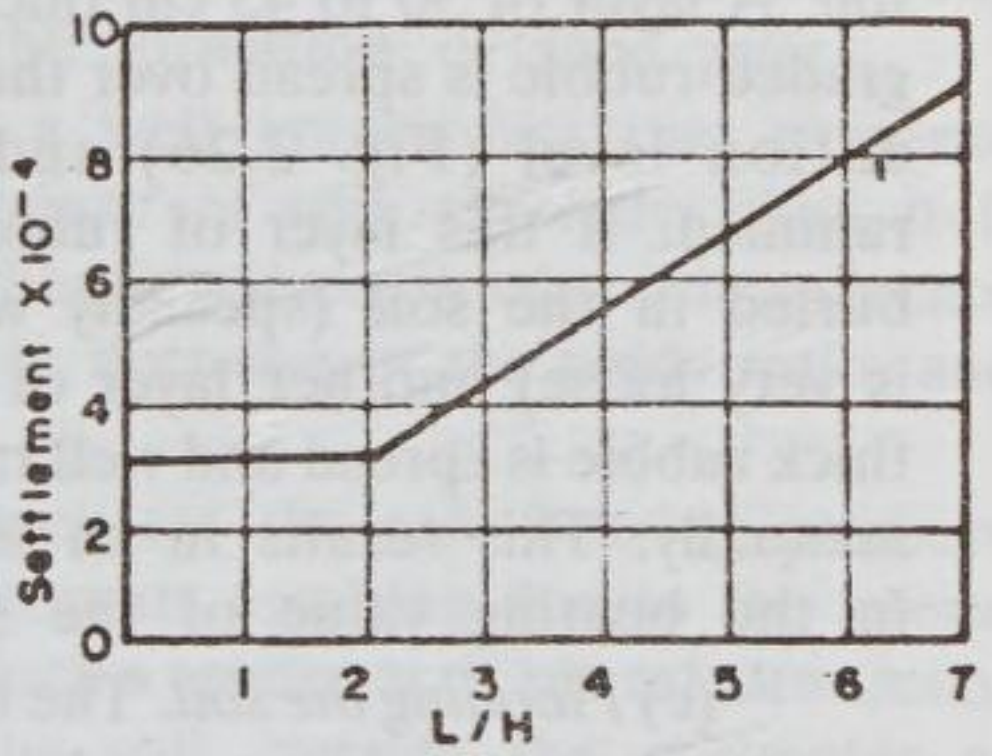


FIG. 2.25. PERMISSIBLE DIFFERENTIAL SETTLEMENT OF BRICK-WALLS.

For $\frac{L}{H} \leq 2$,

Rate of differential settlement

$= 0.0003$ cm/cm

For $\frac{L}{H} = 8$, Rate of differential setlement $= 0.0010$ cm/cm

where L is the wall length and H is the height of wall measured above the base of footing. The rate of differential settlement is defined as the slope or the relative settlement between two points divided by the horizontal distance.

2.13. METHODS OF IMPROVING SAFE BEARING PRESSURE OF SOILS

Sometimes, the safe bearing pressure of soil is so low that the dimensions of the footings work out to be very large and uneconomical. In such a circumstance, it becomes essential to improve the safe bearing pressure, which can be done by the following methods : (*i*) increasing depth of foundation (*ii*) compacting the soil (*iii*) draining the soil (*iv*) confining the soil (*v*) grouting and

(*vi*) chemical treatment.

1. Increasing depth of foundation. It has been found that in granular soil, the bearing capacity increases with the depth due to the confining weight of overlying material. However, this is not economical since the cost of construction increases with the depth. Also, the load on the foundation also increases with the increase in the depth. The method is useful only when better bearing stratum is encountered at greater depth.

2. Compaction of soil. It has found that compaction of *natural* soil deposits (loose) or man-made fills results in the improvement of bearing capacity and reduction in the resulting settlements. Compaction of soil can be effectively achieved by the following means:

(*a*) *Ramming moist soil.* The foundation soil is moistened and then compacted with the help of hand rammers or mechanically operated frog rammers or vibratory rollers. The voids of the soil are very much reduced, resulting in the reduction in settlements.

(*b*) *Rubble Compaction into the soil.* A layer of 30 to 45 cm thick well-graded rubble is spread over the foundation level (Fig. 2.26) and well-rammed. If this layer of rubble gets buried in the soil (specially when it is very loose) another layer of 15 cm thick rubble is spread and well rammed manually. This results in an increase in the bearing value of the soil

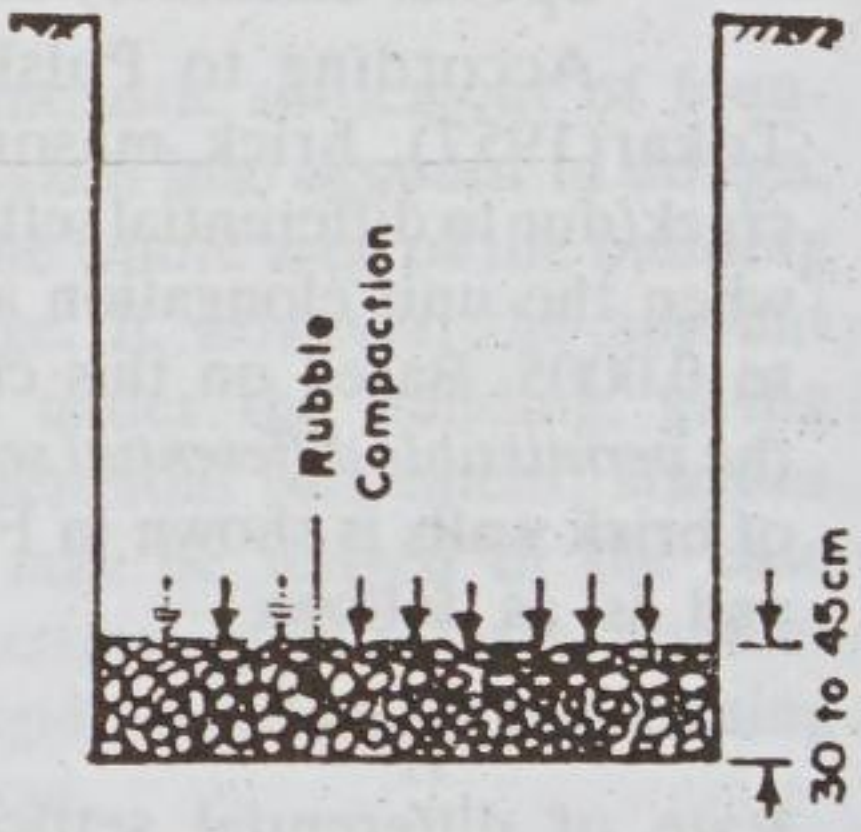


FIG. 2.26. RUBBLE COMPACTION INTO THE SOIL.

(*c*) *Flooding the soil.* The bearing pressure of very loose sands can be increased by flooding the soil. The method is very effective in improving the safe bearing pressure of dune sands, which cannot otherwise be effectively compacted. The Author has an experience of improving the bearing power of desert soils by this method at many locations where it was required to support heavy loads.

(*d*) *Vibration.* Heavy vibratory rollers and compactors may compact a layer of granular soils to a depth of 1 to 3 m. If the method of flooding and then vibration is used, sandy soil can be very effectively compacted, resulting in increased safe bearing power and decreased settlements when super-structure loads come on the soil. After flooding the soil, so that moisture penetration is at least 1 to 2 m, form vibrators or platform vibrators (about 1 m×1.5 m base area, with a pair of eccentrically loaded motors) can be slided on the sand surface with the help of two labourers. A large area can be covered by this process, without the help of sophisticated vibrating equipment.

(*e*)*Vibroflotation.* It is a commercial method which combine the effect of vibration and jetting. A heavy cylinder, known a vibroflot is inserted in the ground (soil) while the cylinder vibrates due to a rotary eccentric weight. A water jet on the tip of the vibro flot supplies a large amount of water under pressure. As the vibro flot sinks, clean sand is added into a crater that develops on the surface. The method is very useful when foundation is required to support heavy loads spread over a greater area.

(*f*) *Compaction by pre-loading.* This method is useful when the footing is founded on clayey soils which result in long term settlements. Pre-loading results in accelerated consolidation, so that settlements are achieved well before the actual footing is laid. The load used for this process is removed before the construction of the footing.

(*g*) *Using sand piles.* This method is very useful in sandy soils or soft soils. Hollow pipes are driven in the ground, at close interval. This results in the compaction of soil enclosed between the adjacent pipes. These pipes are then gradually removed, filling and ramming sand in the hole, resulting in the formation of *sand piles.*

3. Drainage of soil. It is a well known fact that presence of water decreases the bearing power of soil, specially when it is saturated. This is because of low shearing strength of soil in presence of excess water. Drainage results in decrease in the voids ratio, and improvement of bearing power.

4. Confining the soil. Sometimes the safe bearing pressure of the soil is low because of settlements resulting due to the lateral movement of loose granular soil. Such a tendency of lateral movement can be checked by confining the soil, outside the perimeter of foundation area, by driving sheet piles, thus forming an enclosure and confining the soil.

5. Grouting. This method is useful in loose gravels and fissured rocky strata. Bores holes in sufficient numbers are driven in the ground and cement grout is forced through these under pressure. The cracks, voids and fissures of the strata are thus filled with the grout, resulting in the increase in the bearing value.

6. Chemical treatment. In this method, certain chemicals are grouted in the place of cement grout. The chemical should be such that it can solidify and gain early strength.

2.14. CAUSES OF FAILURES OF FOUNDATIONS AND REMEDIAL MEASURES

The foundations may fail due to the following reasons:

1. Unequal settlement of sub-soil. Unequal settlement of the sub-soil may lead to cracks in the structural components and rotation

thereof. Unequal settlement of sub-soil may be due to (*i*) non-uniform nature of sub-soil throughout the foundation, (*ii*) unequal load distribution of the soil strata, and (*iii*) eccentric loading. The failures of foundation due to unequal settlement can be checked by : (*i*) resting the foundation on rigid strata, such as rock or hard moorum, (*ii*) proper design of the base of footing, so that it can resist cracking, (*iii*) limiting the pressure in the soil, and (*iv*)avoiding eccentric loading.

2. Unequal settlement of masonry. As stated earlier, foundation includes the portion of the structure which is below ground level. This portion of masonry, situated between the ground level and concrete footing(base) has mortar joints which may either shrink or compress, leading to unequal settlement of masonry. Due to this, the superstructure will also have cracks. This could be checked by (*i*) using mortar of proper strength, (*ii*) using thin mortar joints, (*iii*) restricting the height of masonry to 1 m per day if lime mortar is used and 1.5 m per day if cement mortar is used, and (*iv*) properly watering the masonry.

3. Sub-soil moisture movement. This is one of the major causes of failures of footings on cohesive soil, where the sub-soil water level fluctuates. When water table drops down, shrinkage of sub-soil takes place. Due to this, there is lack of sub-soil support to the footings which crack, resulting in the cracks in the building. During upward movement of moisture, the soil (specially if it is expansive) swells resulting in high swelling pressure. If the foundation and superstructure is unable to resist the swelling pressure, cracks are induced. For such a situation, special precautionary measures are taken, as discussed in chapter 3.

4. Lateral pressure on the walls. The walls transmitting the load to the foundation may be subjected to lateral pressure or thrust from a pitched roof or an arch or wind action. Due to this, the foundation will be subjected to a moment (or resultant eccentric load). If the foundation has not been designed for such a situation, it may fail by either overturning or by generation of tensile stresses on one side and high compressive stresses on the other side of the footing.

5. Lateral Movement of sub-soil This is applicable to very soft soil which are liable to move out or squeeze out laterally under vertical loads, specially at locations where the ground is sloping. Such a situation may also arise in granular soils where a big pit is excavated in the near vicinity of the foundation. Due to such movement, excessive settlements take place, or the structure may even collapse. If such a situation exists, sheet piles should be driven to prevent the lateral movement or escape of the soil.

6. Weathering of sub-soil due to trees and shrubs. Sometimes, small trees, shrubs or hedge is grown very near to the wall. The roots of these shrubs absorb moisture from the foundation soil, resulting in reduction of their voids and even weathering. Due to this the ground near the wall depresses down. If the roots penetrates below the level of footing, settlements may increase, resulting in foundation cracks.

7. Atmospheric action. The behaviour of foundation may be adversely affected due to atmospheric agents such as sun, wind, and rains. If the depth of foundation is shallow, moisture movements due to rains or drought may cause trouble. If the building lies in a low lying area, foundation may even be scoured. If the water remains stagnant near the foundation, it will remain constantly damp, resulting in the decrease in the strength of footing or foundation wall. Hence it is always recommended to provide suitable plinth protection along the external walls by (*i*) filling back the foundation trenches with good soil and compacting it, (*ii*) providing gentle ground slope away from the wall and (*iii*) providing a narrow, sloping strip of impervious material (such as of lime or lean cement concrete) along the exterior walls.

2.15. SETTING OUT FOUNDATION TRENCHES

Setting out or ground tracing is the process of laying down the excavation lines and centre lines etc. on the ground, before excavation is started. After the foundation design is done, a *setting out plan*, sometimes also known as *foundation layout plan*, is prepared to some suitable scale (usually 1:50). The plan is fully dimensioned.

For setting out the foundations of small buildings, the centre line of the longest outer wall of the building is first marked on

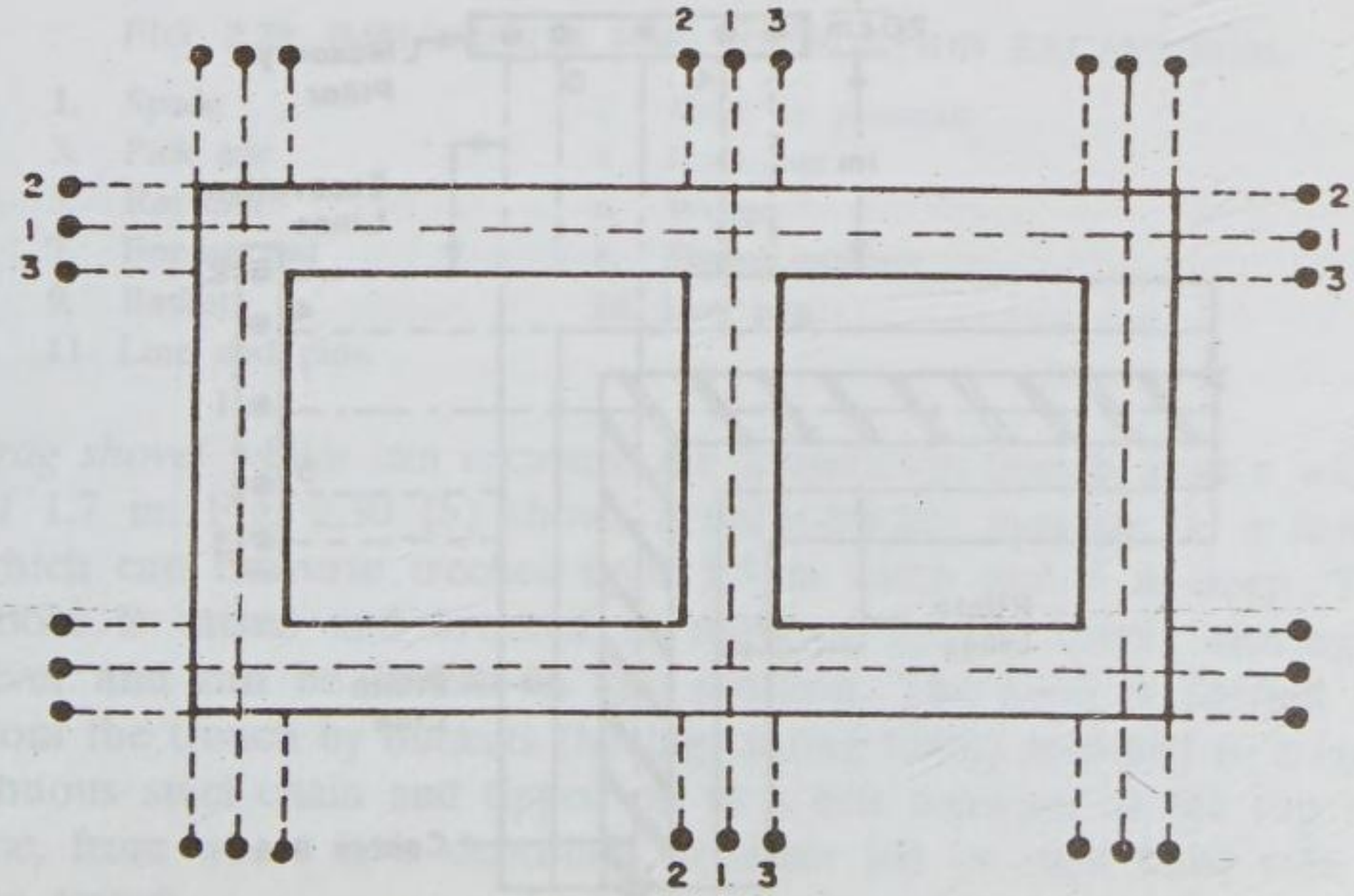


FIG. 2.27. SETTING OUT WITH THE HELP OF PEGS.

the ground by stretching a string between wooden or mild steel pegs driven at the ends. This line serves as reference line. For accurate work, nails can be fixed at the centre of the pegs. Two pegs, one on either side of the central peg, are driven at each end of the line. Each peg is equidistant from the central peg, and the distance between the outer pegs corresponds to the width of foundation trench to be excavated. Each peg may project about 25 to 50 mm above ground level and may be driven at a distance of about 2 m from the edge of excavation so that they are not disturbed.

When string is stretched joining the corresponding pegs (say 2-2) at the two extremities of the line, the boundary of the trench to be excavated can be marked on the ground with dry lime powder. The centre lines of other walls, which are perpendicular to the long wall, are then marked by setting out right angles. A right angle can be set out by forming a triangle with 3, 4 and 5 units long. These dimensions should be measured with the help of a steel tape. Alternatively, a theodolite or prismatic compass may be used for setting out right angles. Similarly, outer lines of the foundation trench of each cross-wall can be set out, as shown in Fig. 2.27.

For a big project, reference pillars of masonry may be constructed as shown in Fig. 2.28. These pillars may be about 20 cm thick, and about 15 cm wider than the width of the foundation trench. The top of the pillars is plastered, and is set at the same level, preferably at the plinth level. Pegs are embedded in these pillars and nails are then driven in the pegs to represent the centre line and the outer lines of the trench. Sometimes, additional walls are provided to represent plinth lines.

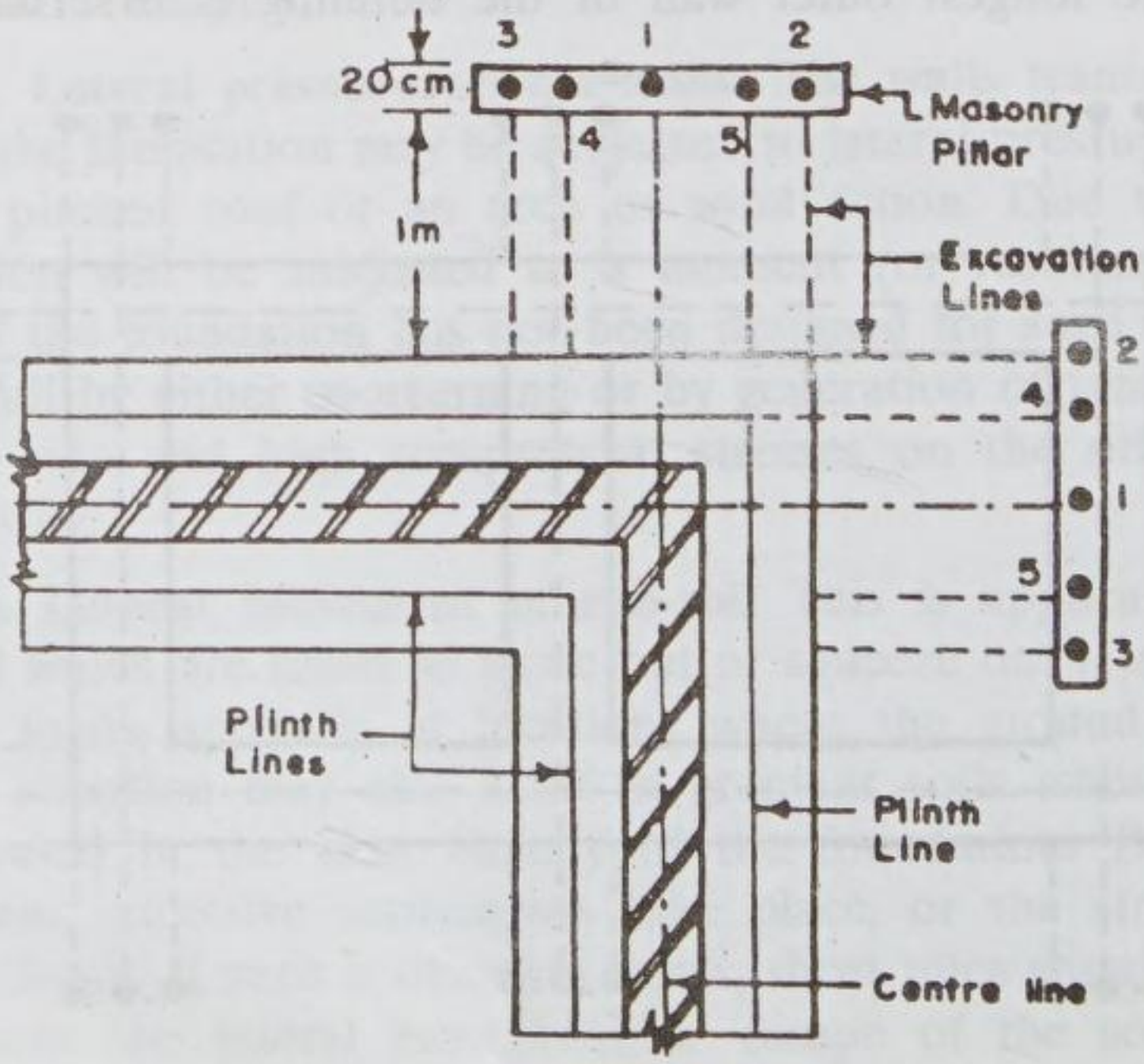


FIG. 2.28. SETTINGS OUT USING MASONRY PILLARS.

2.16. EXCAVATION AND TIMBERING OF FOUNDATION TRENCHES

Excavation of foundation trenches can be done either manually with the help of conventional implements, shown in Fig. 2.29, or with the help of special mechanical equipment. Fig 2.30 (*a*) shows a

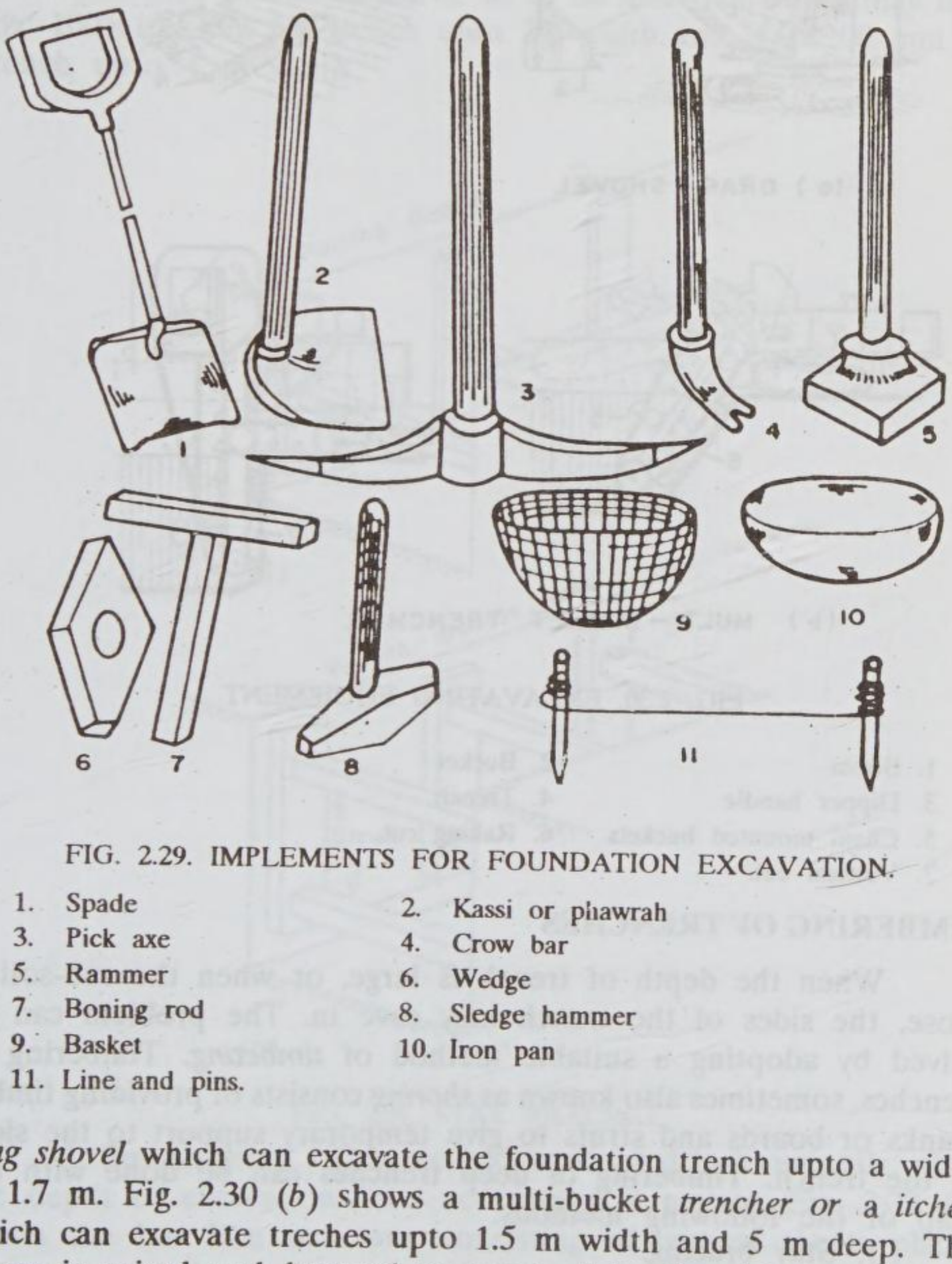


FIG. 2.29. IMPLEMENTS FOR FOUNDATION EXCAVATION.

1. Spade
2. Kassi or phawrah
3. Pick axe
4. Crow bar
5. Rammer
6. Wedge
7. Boning rod
8. Sledge hammer
9. Basket
10. Iron pan
11. Line and pins.

drag shovel which can excavate the foundation trench upto a width of 1.7 m. Fig. 2.30 (*b*) shows a multi-bucket *trencher or* a *itcher,* which can excavate treches upto 1.5 m width and 5 m deep. The boom is raised and lowered as required by the driver moving a lever and can be locked in any position. The spoil is carried up from the trench by buckets (having cutting teeth) attached to a continuous steel chain and tipped on to a belt conveyor at the top the rise, from where it is deposited to either left or right hand side of the trench.

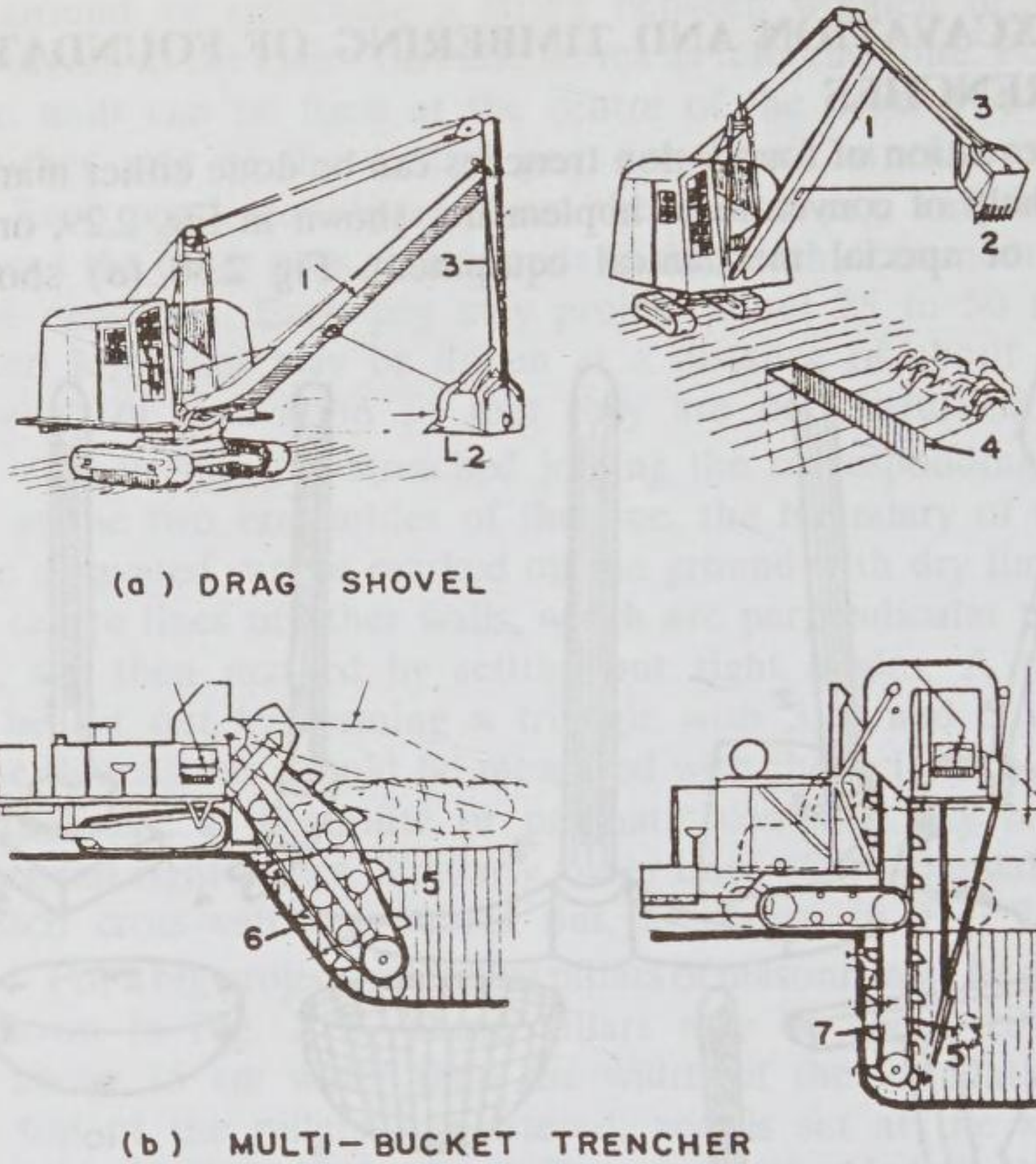


FIG. 2.30. EXCAVATING EQUIPMENT.

1. Boom
2. Bucket
3. Dipper handle
4. Trench
5. Chain mounted buckets
6. Raking cut
7. Vertical cut.

TIMBERING OF TRENCHES

When the depth of trench is large, or when the sub-soil is loose, the sides of the trench may cave in. The problem can be solved by adopting a suitable method of *timbering*. Timbering of trenches, sometimes also known as *shoring* consists of providing timber planks or boards and struts to give temporary support to the sides of the trench. Timbering of deep trenches can be done with the help of the following methods:

1. Stay bracing.
2. Box sheeting
3. Vertical sheeting
4. Runner system
5. Sheet piling.

1. Stay bracing. This method (Fig. 2.31) is used for supporting the sides or a bench excavated in fairly firm soil, when the depth

of excavation does not exceed about 2 metres. The method consists of placing vertical sheets (called sheathing) or polling boards opposite each other against the two walls of the trench and holding them in position by one or two rows of struts. The sheets are placed at an interval of 2 to 4 metres and generally, they extend to the full height of the trench. The polling boards may have width of about 200 mm and thickness of 40 to 50 mm. The struts may have size 100×100 mm for trench upto 2m width and 200×200 mm for trench upto 4 m width.

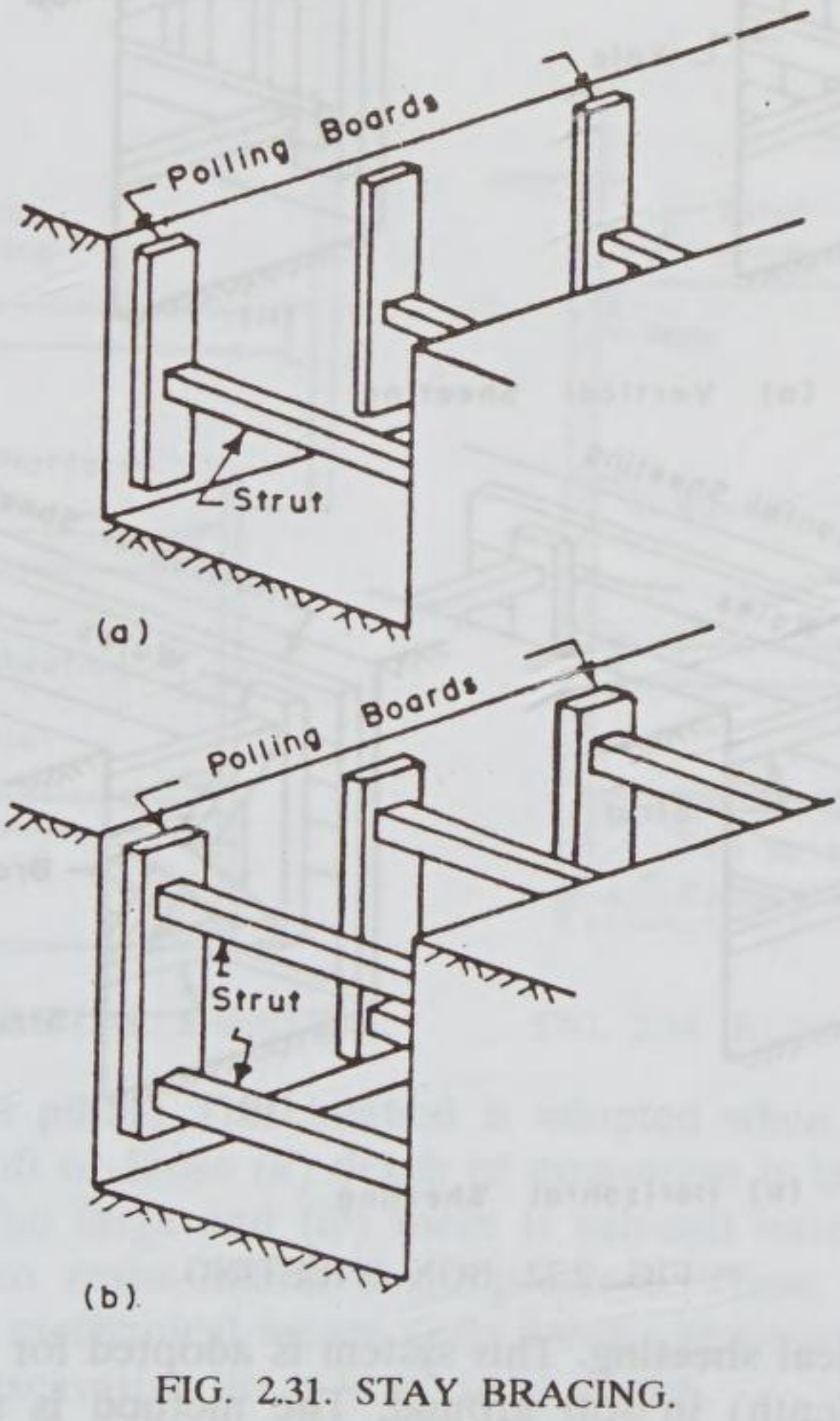


FIG. 2.31. STAY BRACING.

2. Box sheeting . This method is adopted in loose soils, when the depth of excavation does not exceed 4 metres. Fig. 2.32 (*a*) shows the box like structure, consisting of vertical sheets placed very near to each other (some times touching each other) and keeping them in position by longitudinal rows (usually two) of wales. Struts are then provided across the wales.

Another system of box sheeting, shown in Fig. 2.32(*b*), is adopted for very loose soils. In this system, the sheeting is provided longitudinally, and they are supported by vertical wales and horizontal

struts [Fig. 2.32 (*b*)]. If the height is more, braces are also provided along with struts.

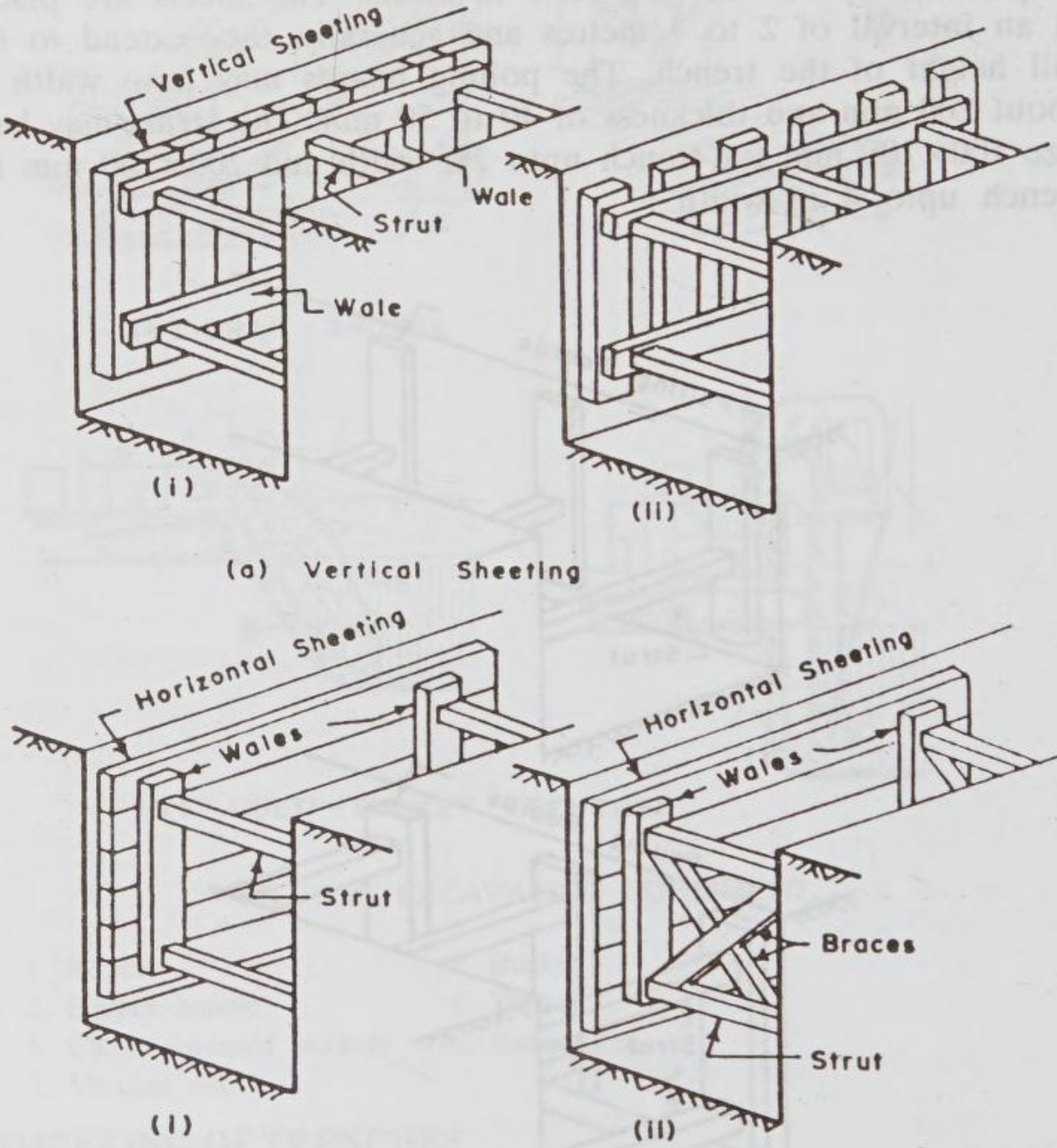


FIG. 2.32. BOX SHEETING.

3. Vertical sheeting. This system is adopted for deep trenches (upto 10 m depth) in soft ground. The method is similar to the box sheeting [Fig. 2.32 (*a*)] except that the excavation is carried out in stages and at the end of each stage, an offset is provided, so that the width of the trench goes on decreasing as the depth increases. Each stage is limited to about 3 m in height and the offset may vary from 25 to 50 cm per stage. For each stage, separate vertical sheeting, supported by horizontal wailings and struts are provided (Fig. 2.33).

4. Runner system. This system is used in extremely loose and soft ground, which needs immediate support as excavation

progresses. The system is similar to vertical sheeting of box system, except that in the place of vertical sheeting, runners, made of long thick wooden sheets or planks with iron shoe at the ends, are provided. Wales and struts are provided as usual (Fig. 2.34). These runners are driven about 30 cm in advance of the progress of the work, by hammering.

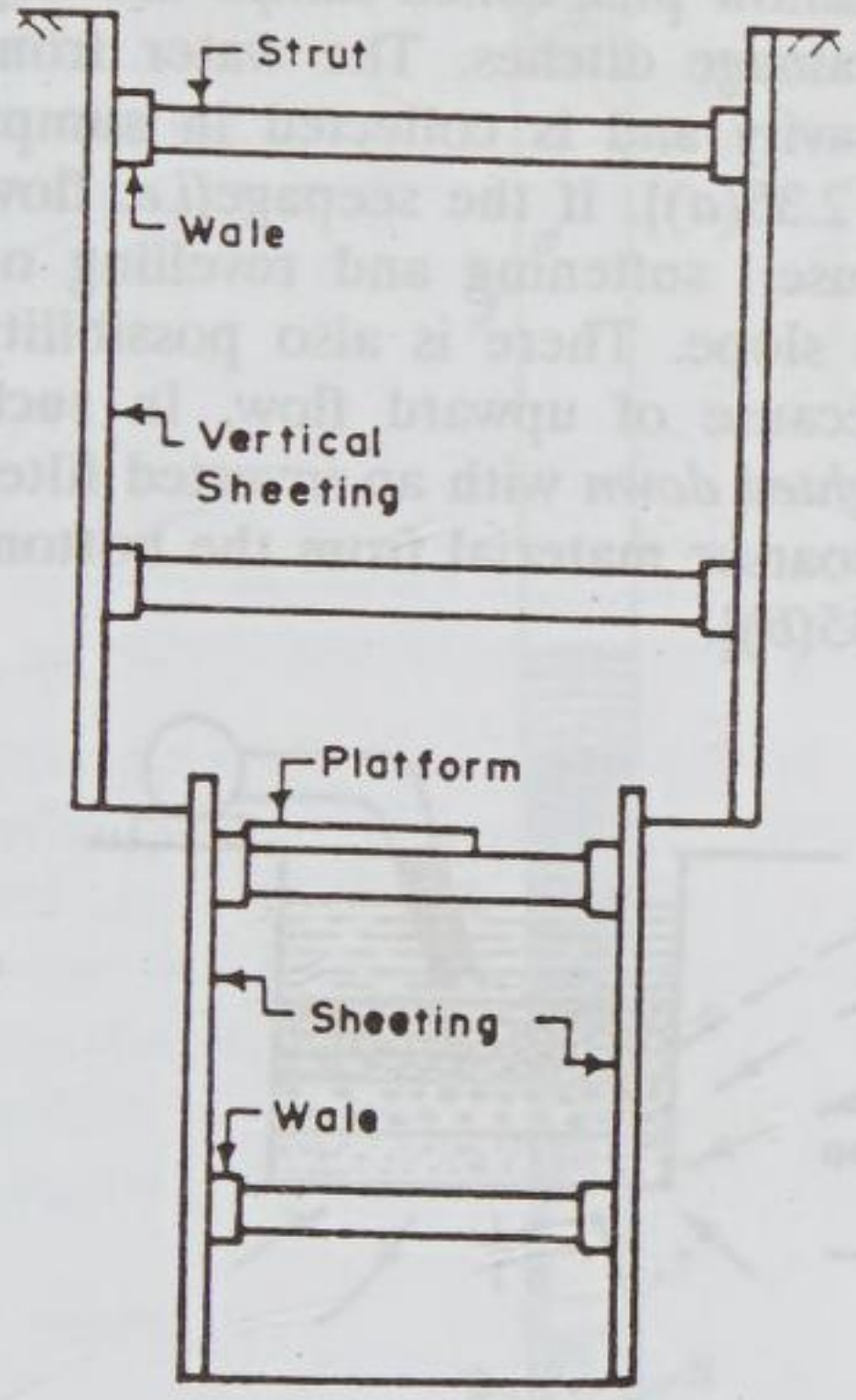


FIG 2.33. VERTICAL SHEETING.

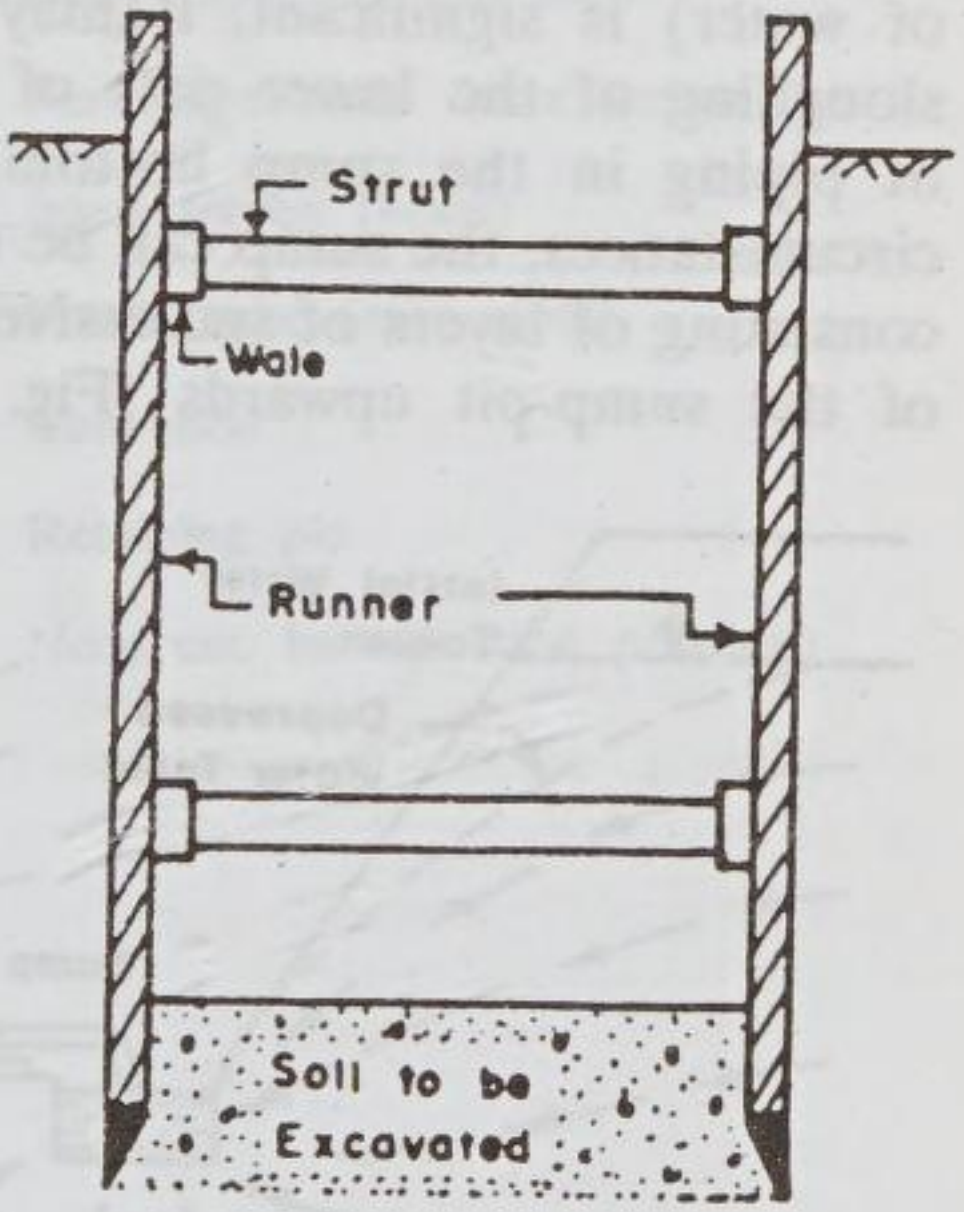


FIG. 2.34. RUNNER SYSTEM.

5. Sheet piling. This method is adopted when (*i*) soil to be excavated is soft or loose (*ii*) depth of excavation is large (*iii*) width of trench is also large and (*iv*) there is sub-soil water. Sheet piles are designed to resist lateral earth pressure. These are driven in the ground by mechanical means (pile driving equipment). They can be used for excavating to a very large depth.

2.17. EXCAVATIONS IN GROUND WITH SUB-SOIL WATER

Excavations of foundation trenches in ground having high water table, or in water-logged area pose great problems because of water oozing in the trench from sides, bringing with it the soil from the sides. The timbering, if provided, would become loose and collapse. Excavations can be carried out by dewatering the sub-soil water. Foundation dewatering can be done by the following methods:

(*i*) Ditches and sumps
(*ii*) Well point system

(*iii*) Shallow well system
(*iv*) Deep well system
(*v*) Vacuum method.
and (*vi*) Electro-osmosis method.

1. Ditches and sumps

This is the simplest form of dewatering used in shallow excavations in coarse grained soils. Shallow pits, called *sumps* are dug along the periphery of the area-drainage ditches. The water from the slopes or sides flows under gravity and is collected in sumps from which it is pumped out [Fig. 2.35(*a*)]. If the seepage(*i.e.* flow of water) is significant, it may cause softening and revelling or sloughing of the lower part of the slope. There is also possibility of piping in the sump bottom, because of upward flow. In such circumstances, the sump can be *weighted down* with an inverted filter consisting of layers of successively coarser material from the bottom of the sump-pit upwards [Fig. 2.35(*b*)].

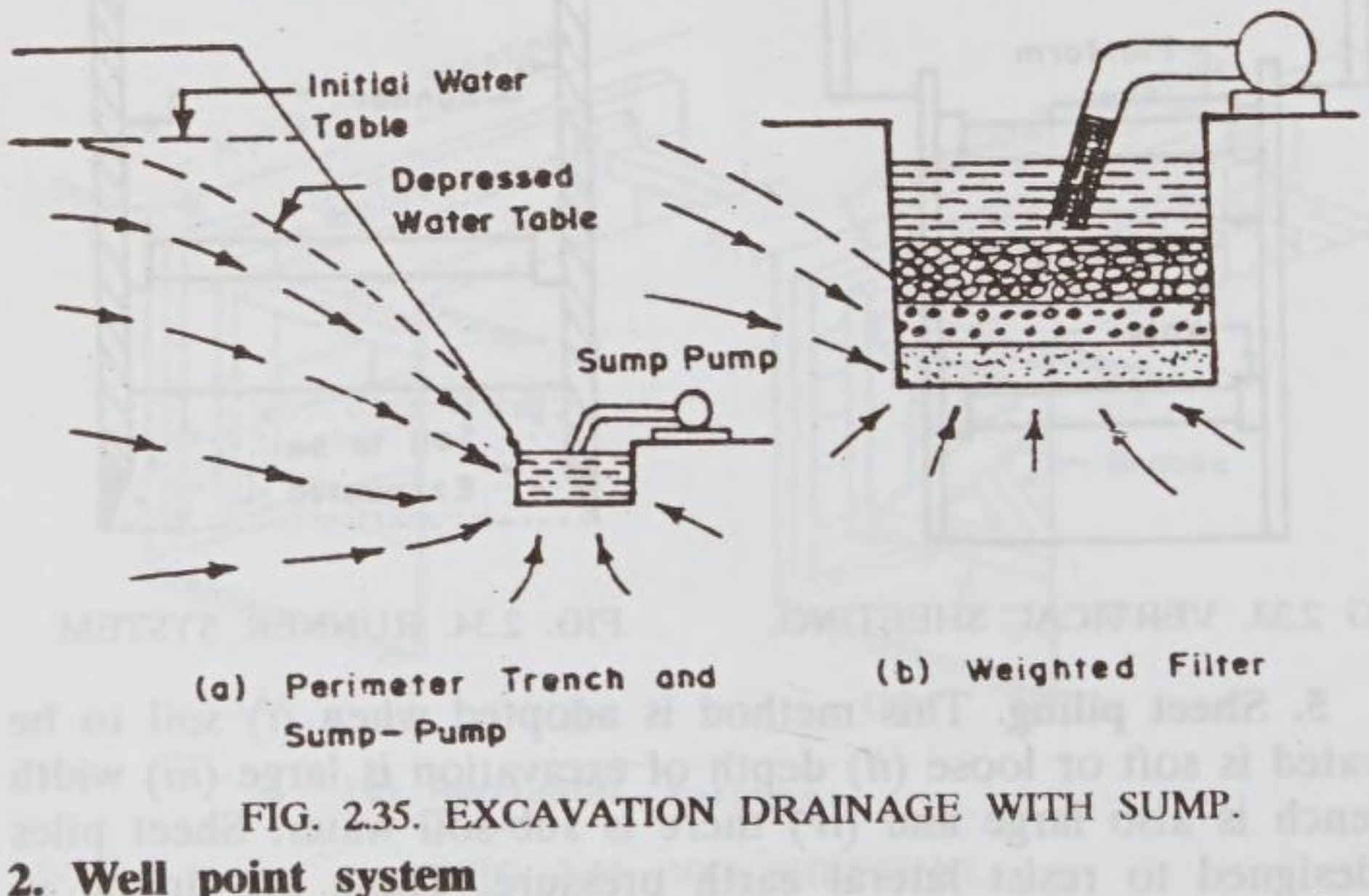
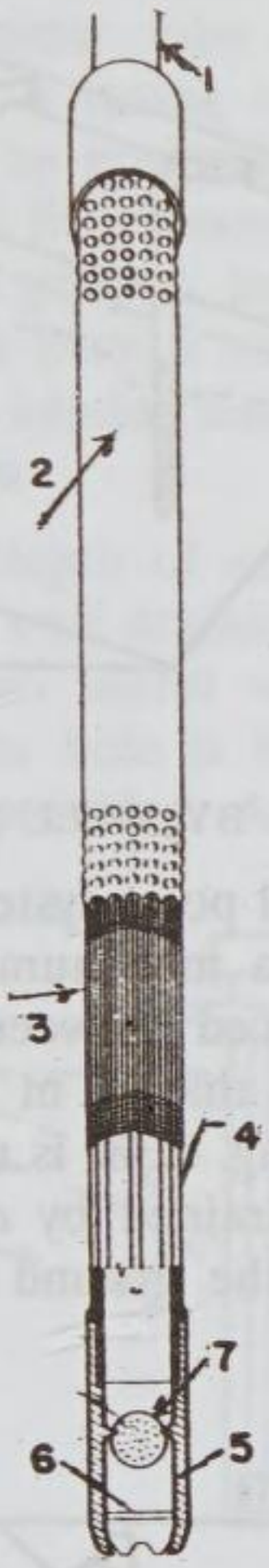


FIG. 2.35. EXCAVATION DRAINAGE WITH SUMP.

2. Well point system

A more complicated dewatering system based on gravity flow is the installation of *well points.* A well point is a perforated pipe, about $\frac{1}{2}$ to 1 m long and 5 to 8 cm in diameter, covered by cylindrical wire gauge screen. In an expensive type well point, the steel tube is covered with two brass screens, the inner of fine mesh and the outer perforated (Fig. 2.36). A conical steel drive point is attached to the lower end of the pipe, with a neoprine ball valve fitted in the point to allow jetting of water to pass through it for driving it. When operating on suction, the ball is in the position shown and the soil water enters the outer screen, through the mesh, and down the flutes of the inner tube. Holes near the bottom of the

latter and just above the shoe admit water to the inside where it is drawn up the riser, along the header to the pump for discharge through the pipes to a drain.



1. Riser pipe (40 mm dia.)
2. Perforated brass outer screen
3. Inner screen (brass)
4. Fluted tube (mild steel)
5. Iron shoe
6. Retaining pin
7. Neoprene ball (suction position)

FIG. 2.36. DETAILS OF WELL POINT

The well points are placed in a row or ring, and the riser pipes are attached through a common manifold or header pipe to a special well point pump (Fig. 2.37). For inserting the well point into ground by jetting, water is pumped down the well point under pressure from where it emerges with a great velocity through the tip of the drive point. The emerging jet-stream dislodges the surrounding soil and the well point can be lowered to the desired depth. A further advantage of jetting is that water under pressure washes away soil fines from around the well point leaving a relatively coarser material to settle and form a natural filter around the well point. The hole formed around the riser pipe and the well point by jetting water is filled with coarse sand. The sand also helps in directing drainage to the well point.

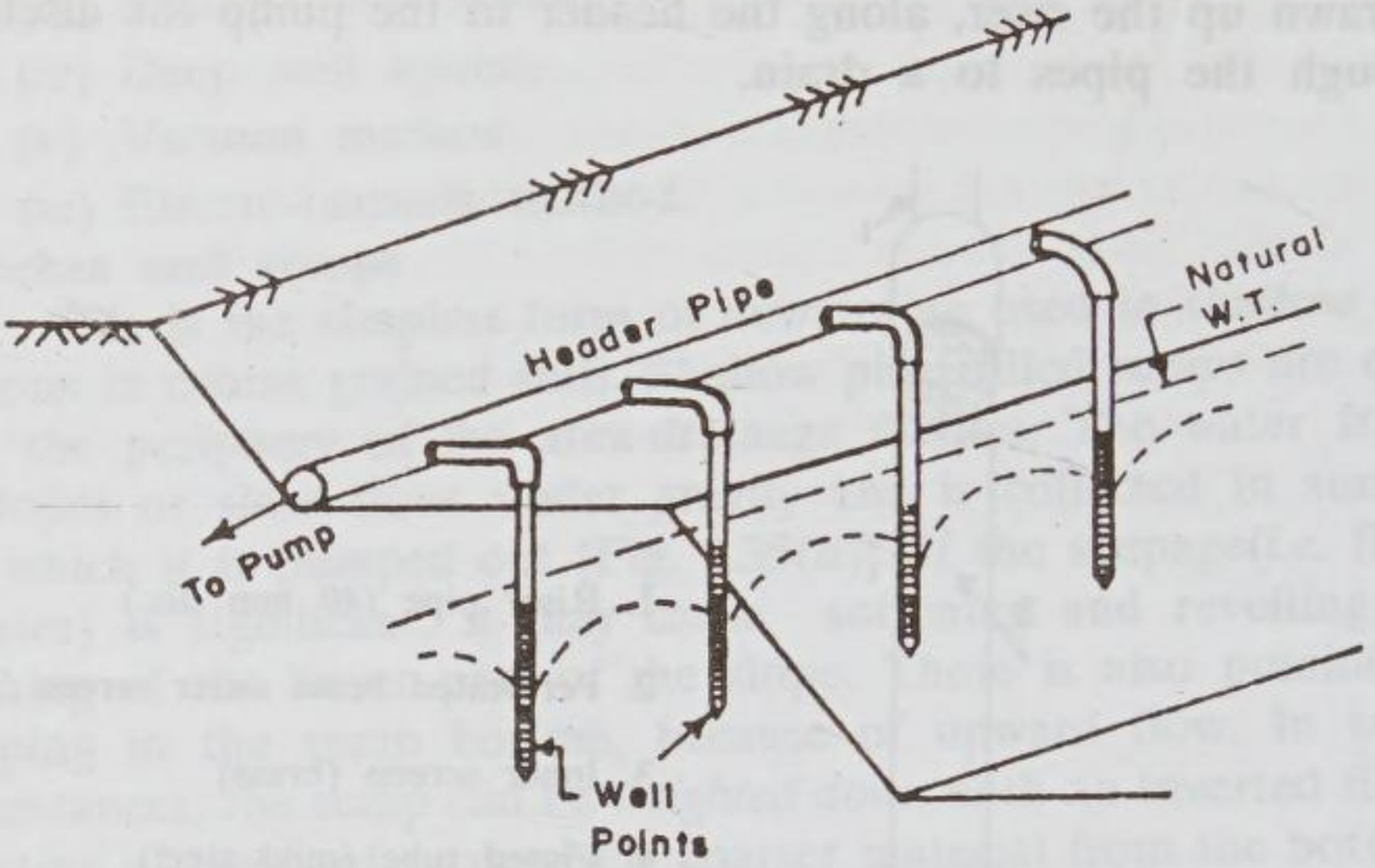


FIG. 2.37. LOWERING OF WATER TABLE BY WELL POINT SYSTEM.

The suction pump used in the well point system has a capacity of bringing water to the surface from a maximum depth of about 6 m. The well points are generally spaced between 1 to 2 m. For dewatering excavations which are more than 6 m below the water table, a multi-stage well point system (Fig. 2.38) is used. Excavations exceeding 16 m depth are preferably drained by *deep well system.* In the multi-stage well point system, the ground is first stripped

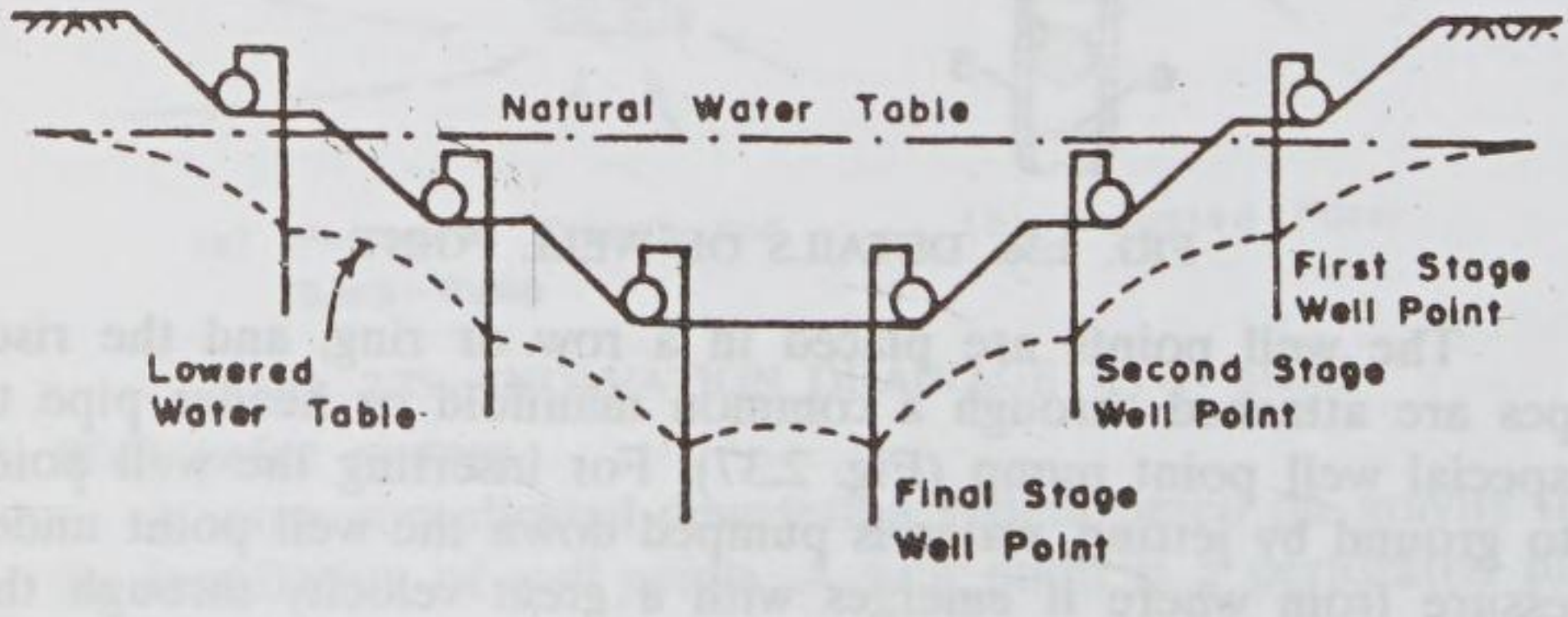


FIG. 2.38. MULTI-STAGE WELL POINT SYSTEM.

to the natural water level where the first stage of well points is installed. After excavating about 5 m, second stage is installed to further lower the water table for advancing excavations. The other stages are put successively, upto a maximum depth of 16 m is reached. In the well point system, a round the clock pumping schedule is essential, as the interruption in pumping can have catastrophic consequences. Hence one auxiliary pumps for each two pumps in use should always be available.

3. Shallow well system

In this system, a hole of 30 cm in diameter or more is bored into the ground to a depth not exceeding 10 m below the axis of the pump. A strainer tube of 15 cm diameter is lowered in the bore hole having a casing tube. A gravel filter is formed around the strainer tube by gradually removing the casing tube and simultaneously pouring filter material, such as gravel etc. in the annular space. A suction pipe is lowered into the filter well so formed. The suction pipes from a number of such wells may be connected to one common header leading to the pumping unit.

4. Deep well system

When the depth of excavation is more than 16 m below the water table, deep well drainage system may be used with advantage. The system is also useful where artesian water is present. A 15 to 60 cm diameter hole is bored and a casing with a long screen (5 to 25 cm) is provided. A submersible pump with a capacity to

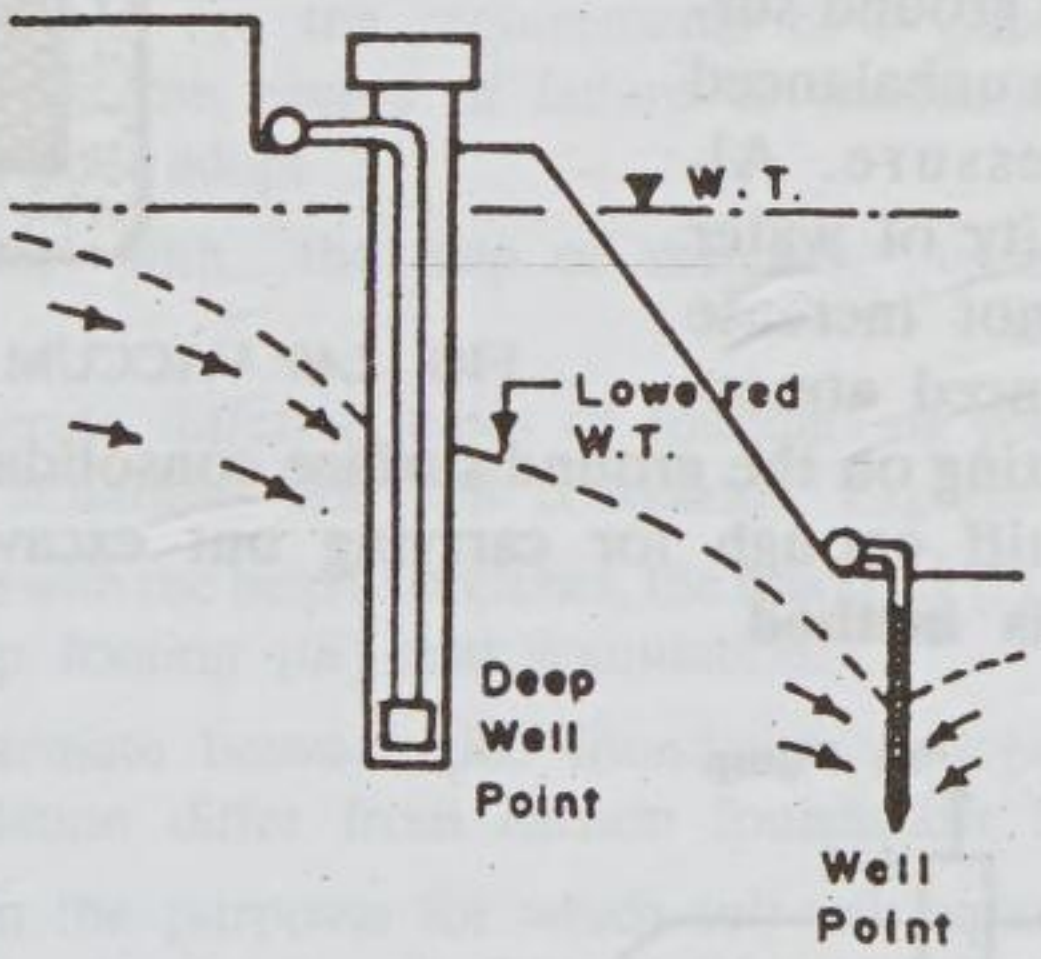


FIG. 2.39. DEEP WELL SYSTEM.

push the water upto a height of 30 m or more is installed near the bottom to the well. Each well has its own pump. Along with the deep wells arranged on the outer side of the area under excavation, a row of well points is frequently installed at the toe of the side slopes of the deep excavation.

5. Vacuum method : Forced flow

The above methods are effective only in coarse grained soils. For fine grained soils, the well point system can be extended by the vacuum method. For successful dewatering in the fine, non-cohesive soils, such as silty sands and other fine sands, it is necessary to apply a suction head to the dewatering system. Both the well point system and deep well system can be adopted for dewatering such

soils by maintaining a vacuum in the well with the use of air tight seals for all points. A hole of about 25 cm diameter is created around the well point and the rise pipe by jetting water under sufficient pressure. While the jetting water is still flowing, medium to coarse sand is rapidly shovelled into the hole to fill it upto about 0.75 or 1 m from the top. The top portion of the hole is then sealed up by tamping bentonite, soil cement or clay (Fig. 2.40). Vacuum pumps are used to create vacuum in the sand filling. When the vacuum is drawn on the well point, the ground surface is subjected to unbalanced atmospheric pressure. Although the quantity of water drawn out does not increase much, the unbalanced atmospheric pressure acting on the ground surface consolidates the sub-soil which becomes stiff enough for carrying out excavations.

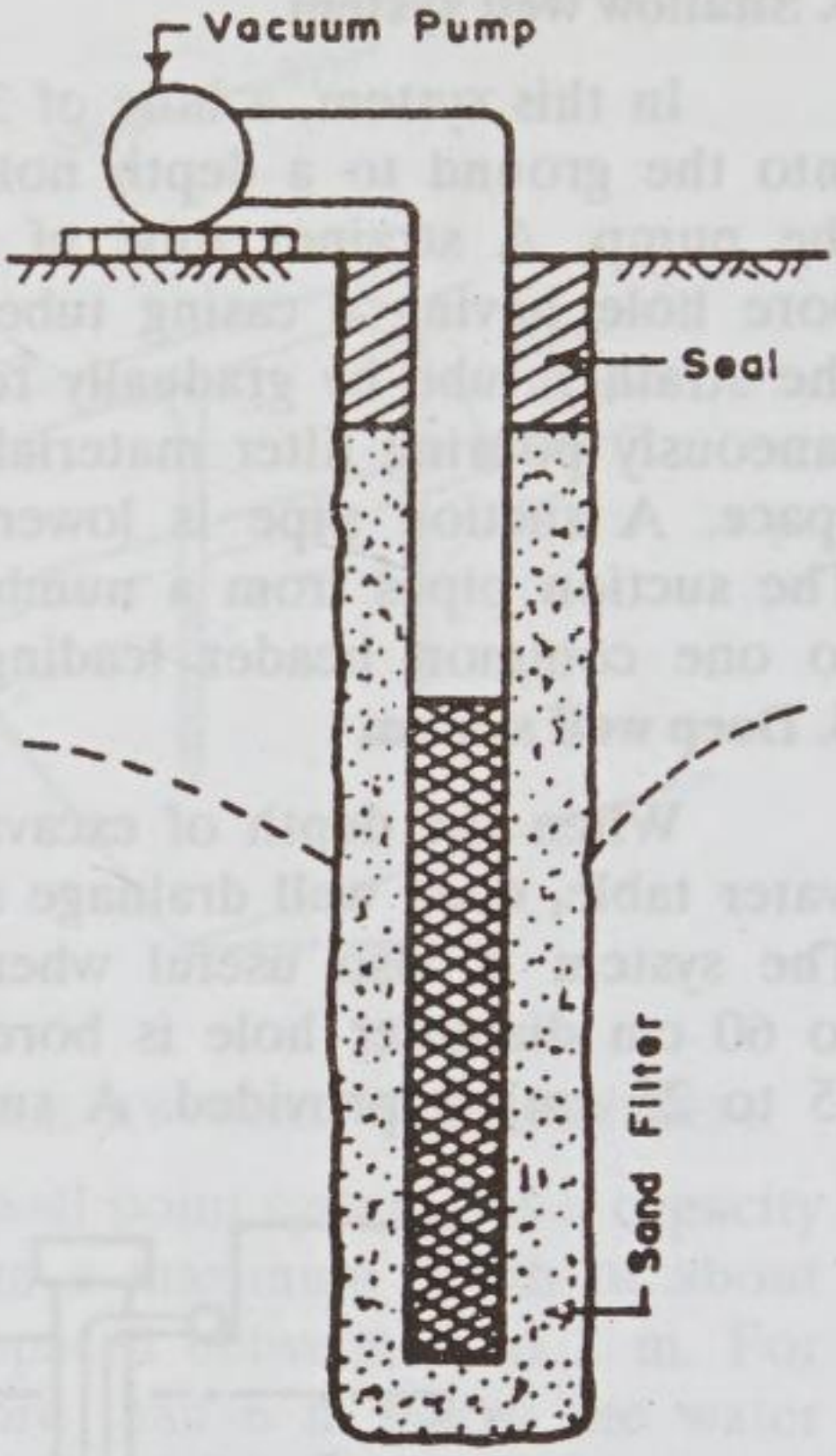


FIG. 2.40 VACCUM METHOD

6. Electro-osmosis method

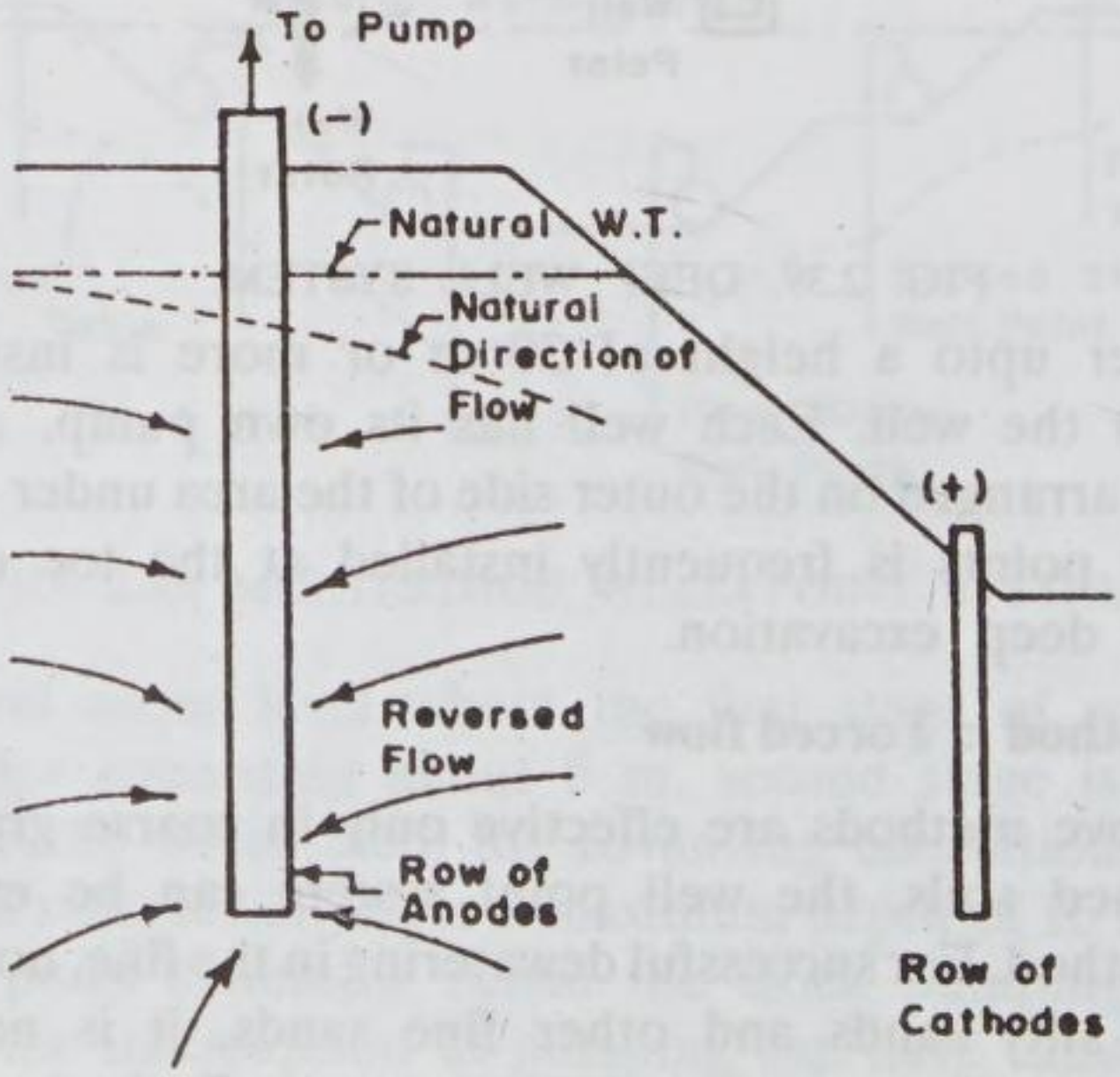


FIG. 2.41. ELECTRO-OSMOTIC DRAINAGE.

This method is used for fine grained cohesive soils (such as clay), which can be drained or stabilised using electric current. The method was developed by L. Casagrande (1952). If direct current is passed between two electrodes driven into natural soil mass, the soil water will travel from the positive electrode (anode) to the negative electrode (cathode). The cathode is made in the form of well point or metal tube for pumping out the seeping form of well point or a metal tube for pumping out the seeping water. A steel rod, a pipe or steel piling of excavation can serve as the cathode. The arrangement of electrodes is done in such a way that the natural direction of flow of water is reversed away from the excavation, thereby increasing the strength of the soil and stability of the slope (Fig 2.41).

The potentials generally used in the process are from 40 to 180 volts, with electrode spacing of 4 to 5 metres.

PROBLEMS

1. (*a*) Discuss various functions served by foundations.
   (*b*) What are the requirements of a good foundation ?
2. What are the causes of failure of foundation ? What remedial measures would you adopt ?
3. Explain, with the help of sketches, various types of shallow foundations.
4. Enumerate different types of foundations you would recommend under different situations and soil conditions. Explain them briefly.
5. Explain with the help of sketches, the following (*i*) trapezoidal combined footing (*ii*) strap footing (*iii*) mat foundation.
6. Differentiate between pile foundation and pier foundation. How does pier foundation differ from caisson foundation ?
7. Explain the purposes for which sub-soil exploration is done. How do you decide the depth of exploration ? Mention the recommended depth of exploration for various types of foundations.
8. What do you understand by a trial pit ? When do you adopt this method ?
9. Explain in brief various methods of boring for sub-soil exploration.
10. What do you understand by a bore hole ? How do you maintain the details of various types of strata obtained during boring ?
11. Write a note on sub-surface soundings.
12. What do you understand by geo-physical methods ? Enumerate various methods used. Which method do you generally use for moderately deep foundations.
13. (*a*) Explain, with the help of sketch, the seismic refraction method.
    (*b*) Explain the electrical resistivity method.

14. Enumerate various methods of sub-soil exploration. What are the factors on which the choice of a particular method depends ?

15 (*a*) Differentiate clearly between disturbed sample undisturbed sample.

(*b*) Explain various types of samplers used for collecting soil samples.

16. Define the following terms: (*i*) ultimate bearing capacity (*ii*) safe bearing capacity (*iii*) allowable bearing pressure. Differentiate clearly between these.

17. (*a*) Enumerate various methods to determine the bearing capacity of soil. Comment on these methods.

(*b*) Derive Rankine's formula for determining the ultimate bearing capacity. How do you use this formula for determining the minimum depth of foundations ?

18. Explain in detail the plate load test for determining safe bearing capacity of soil. Explain the limitations of the test.

19. How do you use settlement criterion for determining safe bearing pressure of soil, using data of plate load test ? Explain how settlement varies with the increase in the bearing area, if the same bearing pressure is assumed.

20. What do youunderstand by penetration tests ? Explain the standard penetration test. How do you use the test data for determining the bearing capacity of soil.

21. Explain the causes of settlement of foundations. What is the value of allowable maximum settlement ?

22. What do you mean by differential settlement ? What are the Code recommendations ? Also, give Polshin and Tokar's recommendations.

23. Explain the methods of improving safe bearing pressure of soils.

24. Describe with sketches the method of setting out foundation trenches.

25. What do you understand by 'timbering' ? Explain with the help of sketches various methods.

26. Enumerate various methods of dewatering foundation excavations. Explain the 'perimeter-ditch-sump method'. What do you understand by weighted filter ?

27. Explain the well point system of foundation dewatering.

28. Write a note on 'electro-osmotic drainage'.

29. Draw typical sketches for foundations for the following :

(*i*) Foundations for square masonry column.

(*ii*) Combined footing for two R.C.C. columns carrying different loads.

(*iii*) Same as above, but one column is situated near the property line.

30. What do you understand by grillage foundation ? Draw a typical sketch for steel grillage foundation for a steel stanchion

3

Foundation -2 : Shallow Foundations

3.1. INTRODUCTION

Foundations may be broadly classified under two heads:

(*a*) Shallow foundations

(*b*) Deep foundations

According to Terzaghi, a foundation is *shallow* if its depth is equal to or less than its width. A shallow foundation is also known as an *open foundation*, since such foundation is constructed by open excavation. Hence those foundations, which have depth even greater than its width, but are constructed by way of open excavation also come under *'shallow foundations'*. A shallow foundation is placed immediately below the lowest part of the super-structure supported by it.

The term *footing* is commonly used in conjunction with shallow foundations. A *footing* is a *foundation unit* constructed in brick work, masonry or concrete under the base of a wall or column for the purpose of distributing the load over a large area.

From the point of view of design, *footings* are classified into four types:

1. Spread footing
2. Combined footing
3. Strap footing

and 4. Mat or raft foundation.

A *Spread footing* is the one which supports either one wall or one column. A spread footing may be of the following types:

1. *Strip footing.* It is the spread footing for wall.
2. *Pad footing.* It is the isolated footing for a column.

When a spread footing supports the load of more than one column or wall, it is called a *combined footing* or *strap footing*. A

mat foundation is a foundation unit continuous in two directions covering an area equal to or greater than the base area of the building.

3.2. DEPTH OF FOOTINGS

The footings should be carried below the top (organic) soil, miscellaneous fill, abandoned foundation, debris or muck. If the top loose soil or fill is too deep, two alternatives may be used depending upon the relative economy and the time available :

(*i*) Removing the top soil directly below the footing and replacing it with lean concrete [Fig. 3.1 (*a*)]

(*ii*) Removing the top soil in an area larger than the footing and replacing it with compacted sand and gravel fill. The area of the compacted sand and gravel fill should be sufficiently large to distribute the footing load, as shown in Fig. 3.1 (*b*).

In either case, it is essential to reach the level of the strata which has the required bearing capacity adopted for the design of footing.

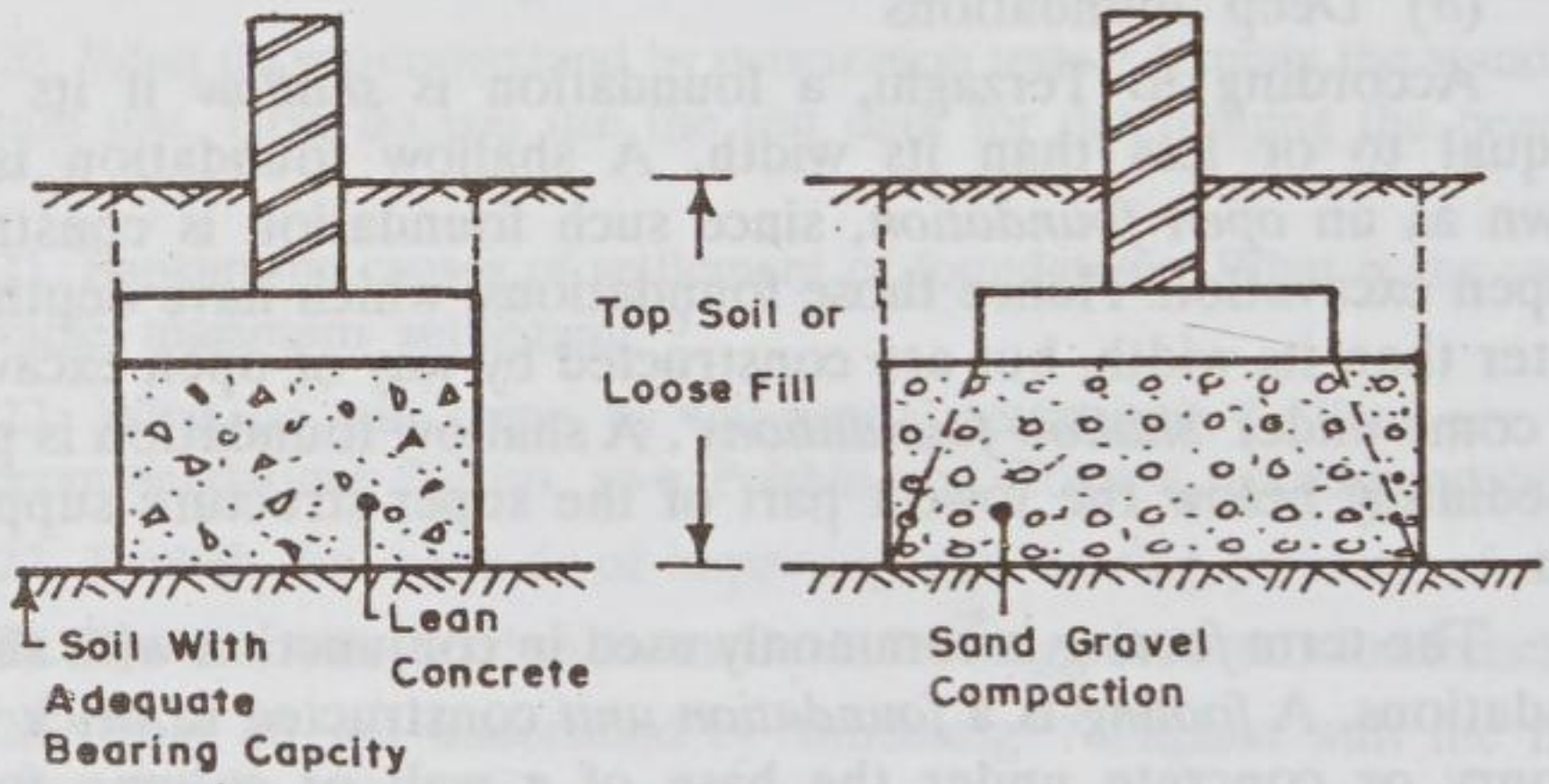


FIG. 3.1.

Sometimes, the top soil may be good and compact, and may have adequate bearing capacity. In that case, it is *desirable* to keep the minimum depth of foundation given by Rankines formula: (Eq.2.8)

$$D_{min} = \frac{q}{\gamma}\left[\frac{1 - \sin\phi}{1 + \sin\phi}\right]^2 \qquad ...(3.1)$$

where ϕ is the *angle of repose*, the values of which may be taken from Table 3.1.

q = Intensity of load at the base of footing (t/m^2 or kN/m^2))

γ = Unit weight of soil (t/m^3 or kN/m^3).

Note : It is to be noted that q is the actual load intensity and *not* the safe bearing capacity of soil. Some times, the actual load intensity may be less than the safe bearing capacity of soil, requiring lesser minimum depth. When footings are supported on very stiff soil, having very high safe bearing capacity, the minimum depth of foundation computed on the basis of safe bearing capacity would come out to be very large which is ridiculous. In such soils, the width of the footing (found from other considerations) would be larger than the one required from the bearing capacity considerations, thus giving rise to actual soil pressure lesser than the safe bearing capacity.

TABLE 3.1. VALUES OF UNIT WEIGHT AND ANGLE OF REPOSE

	Type of soil	*Unit weight (kg/m^3)*	*Angle of Repose*
1.	Sand (dry)	1500 to 1650	25° to 35°
2.	Sand (Damp)	1700 to 1850	30° to 40°
3.	Sand (Wet)	1800 to 1900	15° to 30°
4.	Sand (Dry and compact)	1700 to 1850	35° to 45°
5.	Vegetable earth (Dry)	1600 to 1700	20° to 30°
6.	Vegetable earth (Damp)	1650 to 1750	40° to 45°
7.	Vegetable earth (Wet)	1700 to 1800	15° to 20°
8.	Vegetable earth (Dry and consolidated)	1800	45°
9.	Gravel	1700 to 1800	40° to 45°
10.	Sand-Gravel mix	1800 to 1900	25° to 35°
11.	Clay (Dry)	1700 to 1750	30°
12.	Clay (Damp)	1750 to 1850	35° to 40°
13.	Clay (Wet)	1850 to 1900	15°
14.	Mud	1600 to 1850	zero
15.	Ashes	600 to 800	40°

The depth of footing should also be such that the rate or angle of spread of the load from the wall base to the outer edge of the ground bearing does not exceed the permissible value, as envisaged in Fig. 3.3(*a*).

The National Building Code of India (SP: 7-1970) lays the following recommendations regarding the depth of foundation:

The depth to which foundation shall be carried depends upon (*a*) the securing of adequate bearing capacity, (*b*) the depth of shrinkage and swelling in case of clayes soils, due to seasonal weather changes which are likely to cause appreciable movements, and (*c*) the depth of froast penetration in the case of the fine sand and silts.

All the foundations shall extend to a depth of at least 50 cm below natural ground level. On rock or such other weather-resisting natural ground, removal of top soil may be all that is required.

Where there are conditions adjoining to the subsoil on which the building is to be erected, which are likely to impair the stability of the building, the foundations of the same shall be taken beyond the detrimental influence of such conditions or suitable works shall be constructed for the purpose of shielding from their effects.

3.3. STRIP FOOTING

A strip footing is the one which provides a continuous longitudinal bearing. Thus, a spread footing for a continuous wall is called a strip footing. Fig. 3.2 shows two types of strip footings for a wall : (*a*) simple strip footing without masonry *offset* and (*b*)

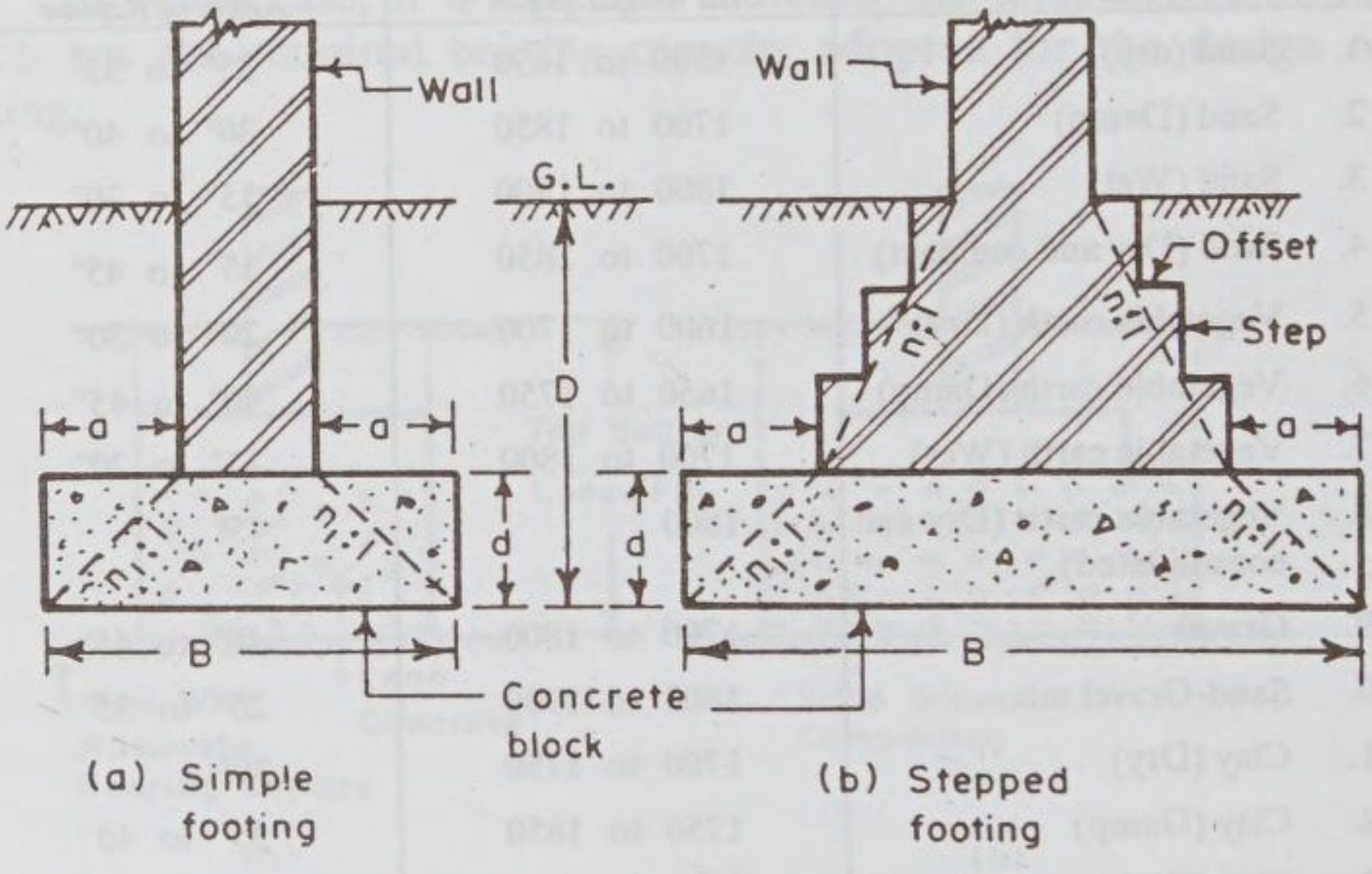


FIG. 3.2. STRIP FOOTING FOR WALL.

strip footing with masonry offsets.

An *offset* is the projection of the lower step from the vertical face of the upper step.

The width of footing is found on the basis of safe bearing pressure for the soil, by expression.

$$B = \frac{W}{q_s} \qquad ...(3.2)$$

where W = Total super-imposed load on the base of the footing

q_s = Safe bearing pressure.

(*a*) **Simple Strip Footing.** (Fig. 3.2 *a*)

When the wall carries light loads or when the safe bearing pressure is very high, the width of the footing found from the above expression would be very small. In that case, a simple strip footing,

such as the one shown in Fig. 3.2 (*a*) is provided. The wall directly rests on the concrete base, and no masonry offsets are provided since spread is not required. However, the concrete base should project out by value *a* on either side of the wall face, where the value of offset *a* may vary from 10 to 20 cm. As a thumb rule, the width of concrete base should not be less than twice the width of the wall. The thickness of concrete block should at least be equal to offset *a* in the case of cement concrete and $\frac{3}{2}a$ in the case of lime concrete base.

National Building Code of India recommends that the angle of spread of the load from the wall base to the outer edge of the ground bearing shall not exceed $n_1 : 1$ (n_1 horizontal and 1 vertical), where $n_1 = 2/3$ for lime concrete and $n_1 = 1$ for cement concrete.

(*b*) Stepped Footing.

When the wall carries heavy loads, or when the safe bearing pressure of the soil is not very high, the base width required from Eq 3.2 will be much greater than $(T + 2a)$. In that case, it is essential to provide masonry offsets, to achieve larger spread, before the load is transferred to concrete base. The height and width of each offset should be so proportioned that rate of spread does not *exceed* the permissible value for the masonry Fig. 3.2(*b*) shows such a stepped footing in which the rate of spread through masonry is n :1 and that through concrete base is $n_1 : 1$. As per National Building Code, the angle of spread of the load from the wall base to the outer edge of the ground bearing shall not exceed the following values:

(*i*) In brickwork and stone masonry : $\frac{1}{2}$ horizontal to 1 vertical

(*ii*) In lime concrete : $\frac{2}{3}$ horizontal to 1 vertical

(*iii*) In cement concrete : 1 horizontal to 1 vertical

Thus, $n = \frac{1}{2}$ and $n_1 = \frac{2}{3}$ (for lime concrete) and $n_1 = 1$ (for cement concrete)

The implication of the above recommendations is that in order to spread the bearing width from original T (width of wall) to B (footing width), the *minimum depth* required would workout as follows:

$$n : 1 :: \frac{B - T}{2} : D_{min}$$

or
$$D_{min} = \frac{1}{2n}(B - T) \qquad ...(3.3)$$

(Assuming uniform rate of spread)

If different rates of spread are taken, and if d is the thickness of concrete block (see Eq. 3.4), we have

$$(B - T) = 2\,[n(D_{min} - d) + n_1 d]$$

or $$D_{min} = \frac{1}{2n}[(B - T) - 2d(n_1 - n)] \quad ...[3.3(a)]$$

Eq. 3.3(a) reduces to Eq. 3.3 when $n_1 = n$.

In the case of brick walls, the offset should not be greater than 5 cm; the corresponding height of each step would work out to be 10 cm. As a thumb rule, the width B' of the bottom brick course should not be less than twice the width of the wall. In the case of stone masonry, the offsets may vary from $7\frac{1}{2}$ cm to 10 cm(max.) corresponding to a min. height of masonry course equal to 15 cm and 20 cm respectively.

Depth of concrete bed block

The depth of concrete bed block depends upon the type of concrete, the projection of the block and the soil bearing pressure. It is found on the basis of the bending moment imposed on it, and on the basis of safe modulus of rupture.

For a footing shown in Fig. 3.3, let

d = depth of concrete block in cm

a = projection beyond the masonry face in cm

m = safe modulus of rupture of concrete mix, in kg/cm^2.

q = net soil bearing pressure, in kg/m^2.

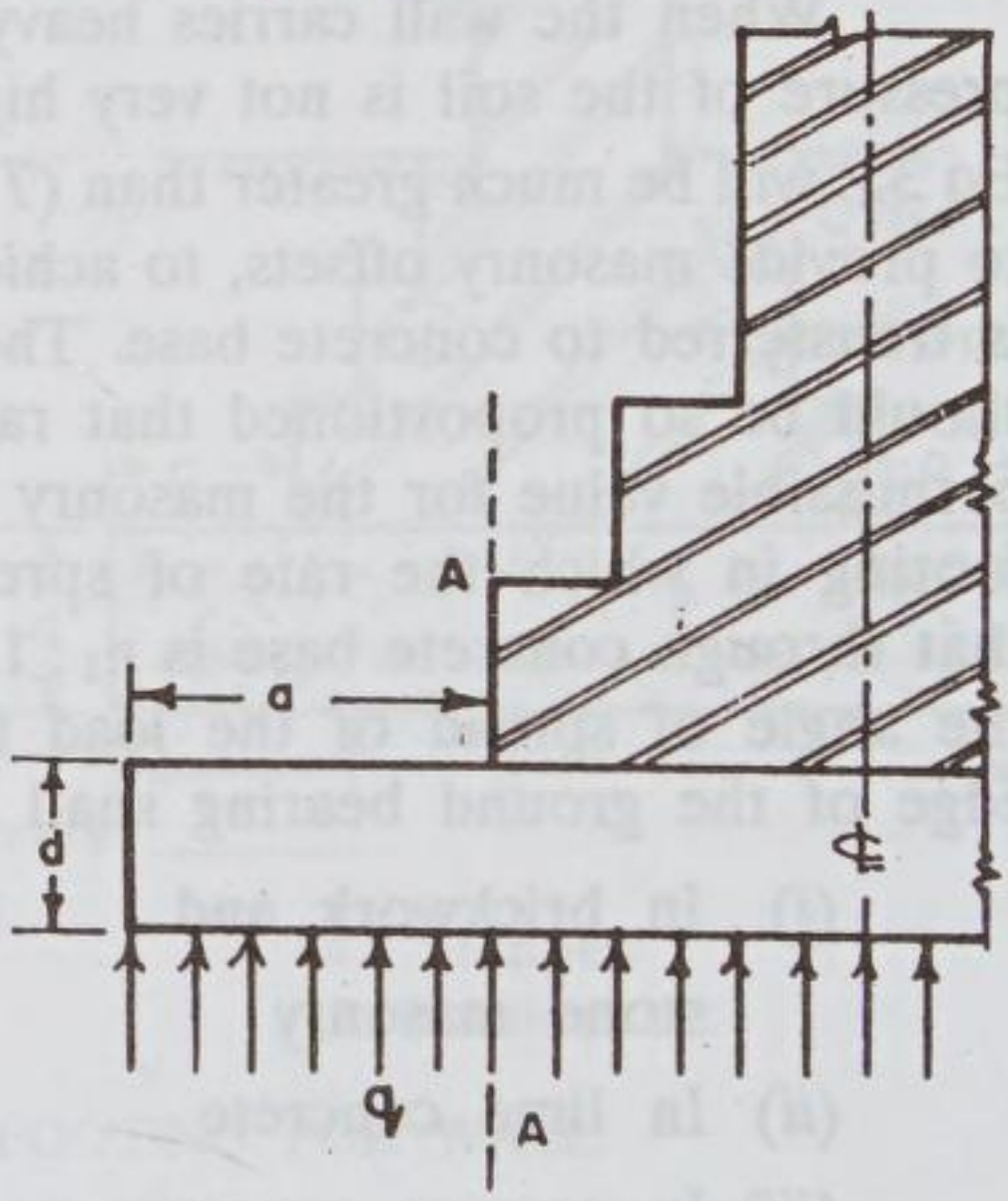


FIG. 3.3.

The projected concrete block will be subjected to bending moment due to upward soil pressure q. The maximum B.M. will be about plane A–A. Consider 1 cm length of the footing (or wall).

$$\text{Soil pressure} = q \text{ kg/m}^2 = \frac{q}{(100)^2} \text{ kg/cm}^2$$

$$\therefore \quad \text{B.M. } M \text{ about } A\text{–}A = \frac{q}{(100)^2} \times \frac{a^2}{2}$$

kg-cm per cm length of footing.

The moment of resistance of the concrete block of 1 cm length is

$$M_r = m\frac{d^2}{6} \text{ kg-cm per cm length.}$$

Equating the two, we get

$$m\frac{d^2}{6} = \frac{q}{10000} \times \frac{a^2}{2}$$

or $$d = \sqrt{\frac{3qa^2}{10000\,m}}$$

or $$d = \frac{a}{100}\sqrt{\frac{3q}{m}}. \qquad ...(3.4)$$

If, however, q and m are expressed in kN/m^2, and a and d are in metres:

B.M. M about A–$A = q\frac{a^2}{2}$ N-m per m length of footing

The *moment of resistance* of the concrete block of 1 m length is

$$M_r = m\frac{d^2}{6} \quad \text{kN-m per m length}$$

Equating the two,

$$\frac{md^2}{6} = q\frac{a^2}{2}$$

or $$d = \sqrt{\frac{6qa^2}{2m}}$$

From which $$d = a\sqrt{\frac{3q}{m}} \qquad ...[3.4\ (a)]$$

In the above expression, q and m are in the same units (kN/m^2) while d and a are also in the same units(*i.e.* either in metres or in cm or in mm).

Equation 3.4 (a) is also valid if both q and m are expressed in t/m^2 units.

The value of modulus of rupture m for various types of concrete are given in Table 3.2.

TABLE 3.2. SAFE MODULUS OF RUPTURE OF CONCRETE

S.No.	*Type of concrete*	*Modulus of rupture*		
		Kg/cm²	*t/m²*	*kN/m²*
1.	Lime concrete	1.54	15.4	154
2.	Cement concrete (1 : 2 : 4)	5.25	52.5	525
3.	Cement concrete (1 : 3 : 6)	3.50	35.0	350
4.	Cement concrete (1 : 4 : 8)	2.45	24.5	245

3.4. ISOLATED FOOTING OR PAD FOOTING

A spread footing for a single column is either known as the *isolated footing* or *pad footing*. The base area A of such a footing is given by

$$A = \frac{P}{q_s} \qquad ...(3.5)$$

where P is the total load transmitted by the column, including that of the footing and q_s is the safe bearing pressure for the soil.

(a) Simple pad footing.

If P is small, or q_s is large, A will also be small. In that case, the footing may consist of simple concrete block projecting out from the column face on all sides. The shape of the footing is generally kept the same as that for the column (*i.e.* trapezoidal, square or circular), as illustrated in Fig. 3.4, so that equal projection ($=a$) is obtained for the base concrete.

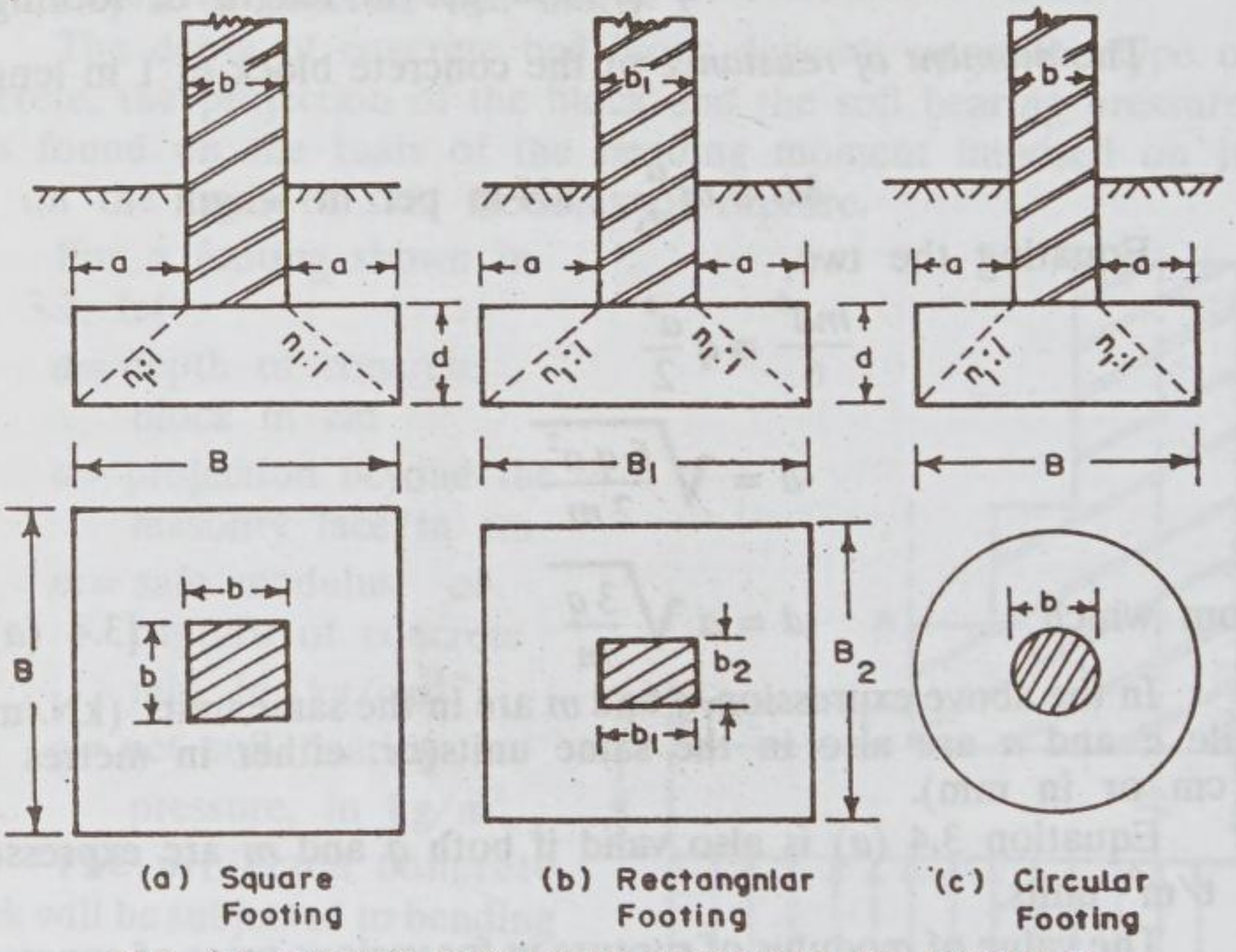


FIG. 3.4. SIMPLE ISOLATED FOOTINGS OF LIGHTLY LOADED COLUMNS.

The value of offset a may vary from 10 to 20 cm. As a thumb rule, the base dimensions of the concrete base should not be less than twice the appropriate lateral dimension of the column in that direction. The thickness of concrete block should atleast be equal to a in the case of cement concrete and $\frac{3}{2}a$ in the case of lime concrete base. National Building Code of India recommends that the angle of spread of load shall not exceed $n_1 : 1$ where $n_1 = \frac{2}{3}$ for lime concrete and $n_1 = 1$ for cement concrete.

(b) Stepped pad footing.

If the column load is more , or if the safe bearing pressure of the soil is less, the base area found by Eq 3.5 will be large (much greater than $b + 2a$). In that case, it is necessary to provide

masonry offsets, to achieve larger spread, before the load is transferred to the concrete base. The height and width of each offset shold be so proportioned that rate of spread does not exceed the permissible value for the masonry. Fig. 3.5 shows the stepped footing, in which the rate of spread is $n:1$ for masonry and $n_1:1$ for concrete, where $n = \frac{1}{2}$ for masonry, and $n_1 = 2/3$ for lime concrete and 1 for cement concrete.

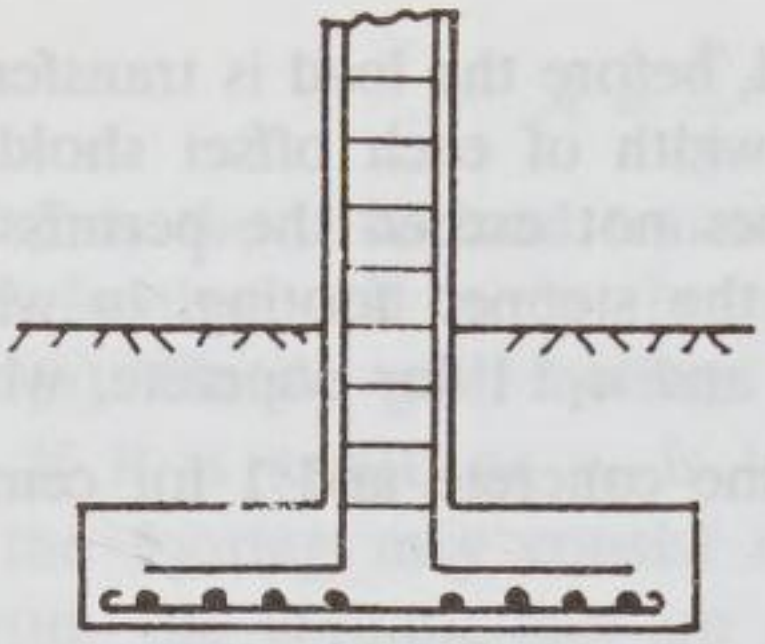
In the case of brick pillar, the offsets should not exceed 5 cm. In the case of masonry pillar, the offset may vary between 10 to 15 cm corresponding to the step height of 15 to 22.5 cm respectively.

FIG. 3.5. STEPPED PAD FOOTING.

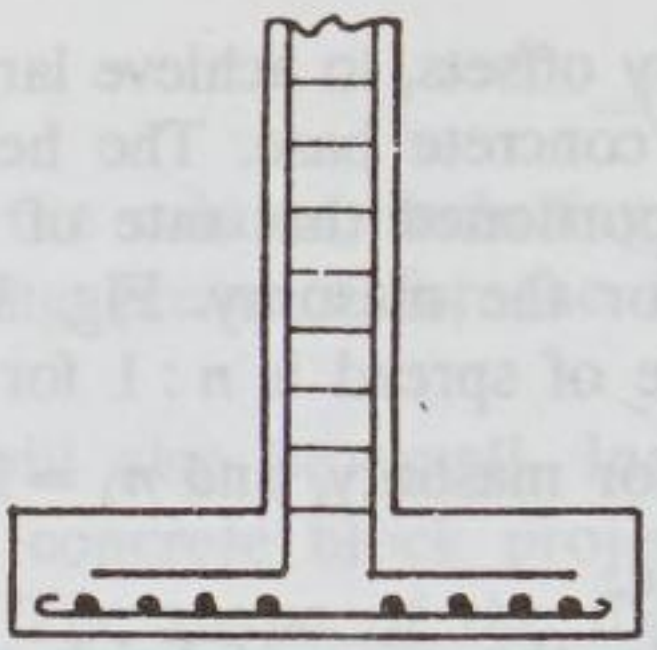
The depth of concrete block is given by

$$d = a\sqrt{\frac{3q}{m}} \qquad ...(3.6)$$

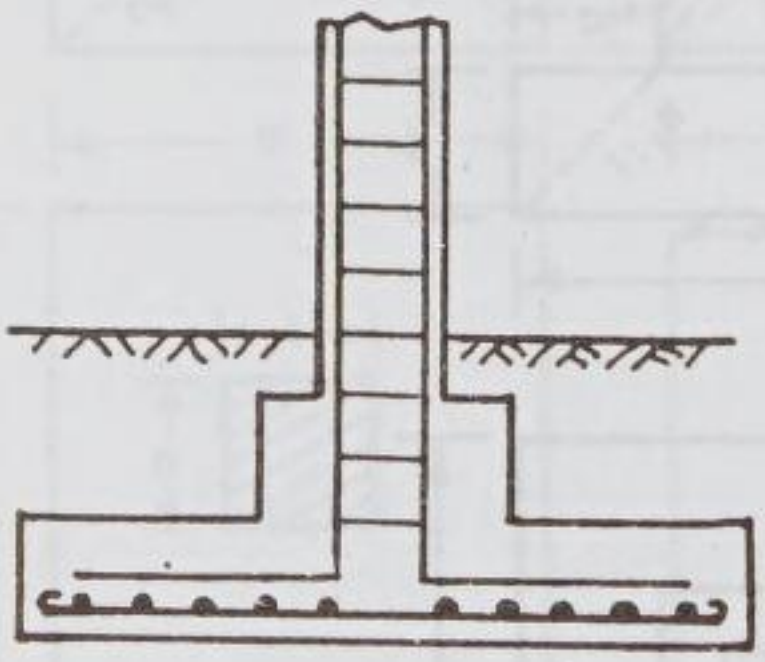
where q and m are in the same units (*i.e.* in kg/m^2 or t/m^2 or kN/m^2), and d and a are in the same units (*i.e.* in cm or m or mm).



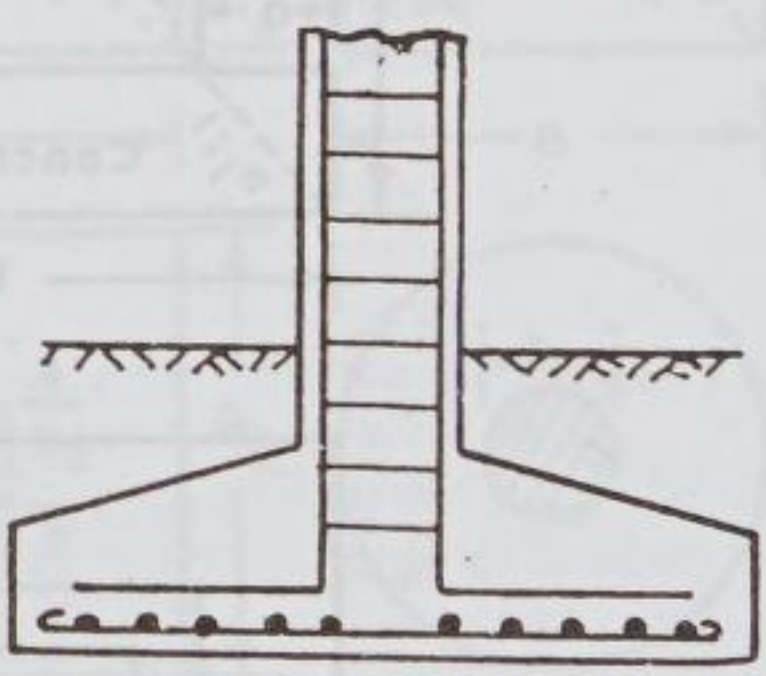
(a) Square Footing



(b) Circular Footing



(c) Square Footing With Padestal.



(d) Sloped Footing

FIG. 3.6. REINFORCED CONCRETE FOOTINGS FOR R.C.C. COLUMNS.

(c) Footings for reinforced concrete columns.

Reinforced concrete columns are supported on reinforced concrete footings only. Fig. 3.6 shows typical details of such footings.

For structural design of these footings, the reader may refer to Author's book 'Reinforced Concrete Structures (Vol. I).

3.5 ECCENTRICALLY LOADED FOOTINGS

Normally, the footings are so designed and proportioned that the C.G. of the superimposed load coincides with the C.G. of the base area, so that the footing is subjected to concentric loading, resulting in uniform bearing pressure. However, in some cases, it may not be possible to do so. For example, if the wall (or column) under construction is near some other property, it will not be possible to spread the footing to both the sides of the wall or column. Such a situation is shown in Fig. 3.7.

Let W_1 = Super-imposed load, including the weight of wall, per unit length

W_2 = Weight of foundation.

W = Resultant load on the base.

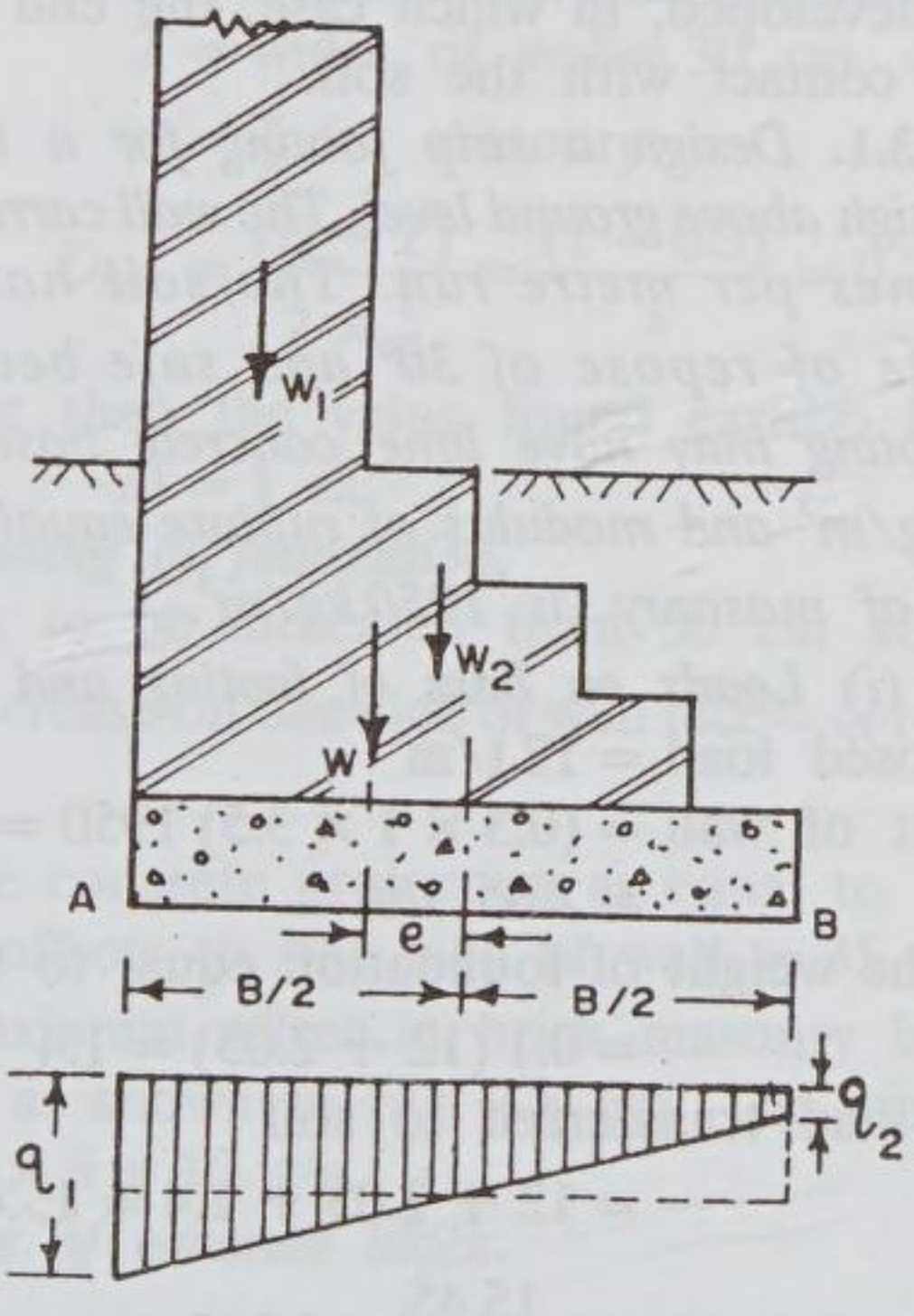


FIG. 3.7. ECCENTRIC LOADING.

Let this resultant load have an eccentricity e with respect to the centre of base width B. This eccentric weight is equivalent to (*i*) a centrally placed load W and (*ii*) bending moment $M = W \cdot e$.

Due to these two, a trapezoidal soil pressure diagram, having pressure intensities q_1 and q_2 will result.

$$q_1 = \frac{\text{load}}{\text{area}} + \frac{\text{B.M.}}{\text{section modulus}}$$

$$= \frac{W}{B} + \frac{W.e}{B^2/6} = \frac{W}{B}\left(1 + \frac{6e}{B}\right) \quad ...[3.7(a)]$$

and $$q_2 = \frac{\text{load}}{\text{area}} - \frac{\text{B.M.}}{\text{section modulus}} = \frac{W}{B} - \frac{W.e}{B^2/6}$$

$$= \frac{W}{B}\left(1 - \frac{6e}{B}\right) \quad ...[3.7(b)]$$

The magnitude of q_1 should not exceed the safe bearing pressure for the soil. Also, in order that the footing may remain in contact with soil, q_2 should be positive (*i.e.* no tension should be developed). In the extreme case, $q_2 = 0$, when $e = \frac{B}{6}$. This gives the maximum value of eccentricity. In that case, $q_1 = \frac{W}{B}(1 + 1) = \frac{2W}{B}$ $= 2 \times$ average pressure on the foundation. If e is greater than $B/6$, tension will be developed, in which case, the end B of the footing will have loose contact with the soil.

Example 3.1. *Design a strip footing for a brick wall 30 cm thick, and 3.5 m high above ground level. The wall carries a superimposed load of 12 tonnes per metre run. The soil has unit weight of 1700 kg/m³ angle of repose of 30° and safe bearing capacity of 16 t/m². The footing may have lime concrete base, which has unit weight of 2000 kg/m³ and modulus of rupture equal to 16 t/m². Take the unit weight of masonry as 1950 kg/m³.*

Solution. (*i*) ***Loads on base of footing and footing width***

Super-imposed load = 12 t/m

Self weight of wall = (0.3 × 1 × 3.5) 1950 = 2050 kg/m
= 2.05 t/m

Assume the weight of foundation equal to 10% of total load
= 0.1 (12 + 2.05) ≈ 1.4 t/m.

∴ Total load transferred to soil
= 12 + 2.05 + 1.4 = 15.45 t/m.

∴ Width of footing $= \frac{15.45}{16} = 0.965$ m.

Provide $B = 1$ m.

(*ii*) ***Depth of footing***

The minimum depth of footing is given by Rankines formula.

$$D_{min} = \frac{q}{\gamma}\left[\frac{1 - \sin\phi}{1 + \sin\phi}\right]^2$$

where q = soil pressure $= \frac{15.45}{1} = 15.45$ t/m² ,

γ = unit weight of soil = 1700 kg/m^3 = 1.7 t/m^3,
ϕ = angle of repose = 30°

$$\therefore \quad D_{min} = \frac{15.45}{1.7}\left(\frac{1-\sin 30°}{1+\sin 30°}\right)^2 \approx 1 \text{ m.}$$

However, the minimum depth is also governed by Eq. [3.3(*a*)] which is based on the requirement that the angle of spread of load should not exceed the permissible values.

$$\therefore \quad D_{min} = \frac{1}{2n}[(B-T) - 2d(n_1 - n)]$$

For computation of D_{min}, either we have to assume some suitable value of d (*i.e.* thickness of concrete block), or n_1 may be assumed to be equal to n. In the latter case, we have

$$D_{min} = \frac{B-T}{2n}$$

where B = width of footing = 1 m,
T = width of wall = 30 cm = 0.3 m,
$n = \frac{1}{2}$ for masonry

$$\therefore \quad D_{min} = \frac{(B-T)}{2 \times \frac{1}{2}} = \frac{(1-0.3)}{1} = 0.7 \text{ m.}$$

This is lesser than the value found earlier. Hence adopt

$$D = 1 \text{ m.}$$

(*iii*) ***Proportioning of foundation.***

The width is to be increased from 30 cm at ground level to 100 cm at base. Increase on one side of wall face = $\frac{1}{2}(100 - 30) = 35$ cm.

Let us fix the concrete projection as equal to 15 cm. Hence the total width of offsets to one side of wall = 35 − 15 = 20 cm.

Since the maximum offset in brick masonry is 5 cm, there will be four offsets as shown in Fig. 3.8 (*a*). The minimum height of each offset = 2 × 5 = 10 cm.

(*iv*) ***Thickness of concrete block.***

Offset a = 15 cm.

Thickness of concrete block is given by

$$d = a\sqrt{\frac{3q}{m}}.$$

where q = bearing pressure on soil = 15.45 t/m^2
m = safe modulus of rupture for lime concrete = 16 t/m^2

$$\therefore \quad d = 15\sqrt{\frac{3 \times 15.45}{16}} = 25.5 \text{ cm}$$

Provide $d = 30$ cm.

Note. If the concrete block is provided in 1 : 2 : 4 cement concrete, having $m = 52\ t/m^2$ (say),

$$d = 15\sqrt{\frac{3 \times 15.44}{52}} = 14.2 \text{ cm.}$$

However, a minimum depth $d = 15$ cm has to be provided since the spread of the load cannot be steeper than 1 : 1 (*i.e.* $n_1 = 1$ for cement concrete).

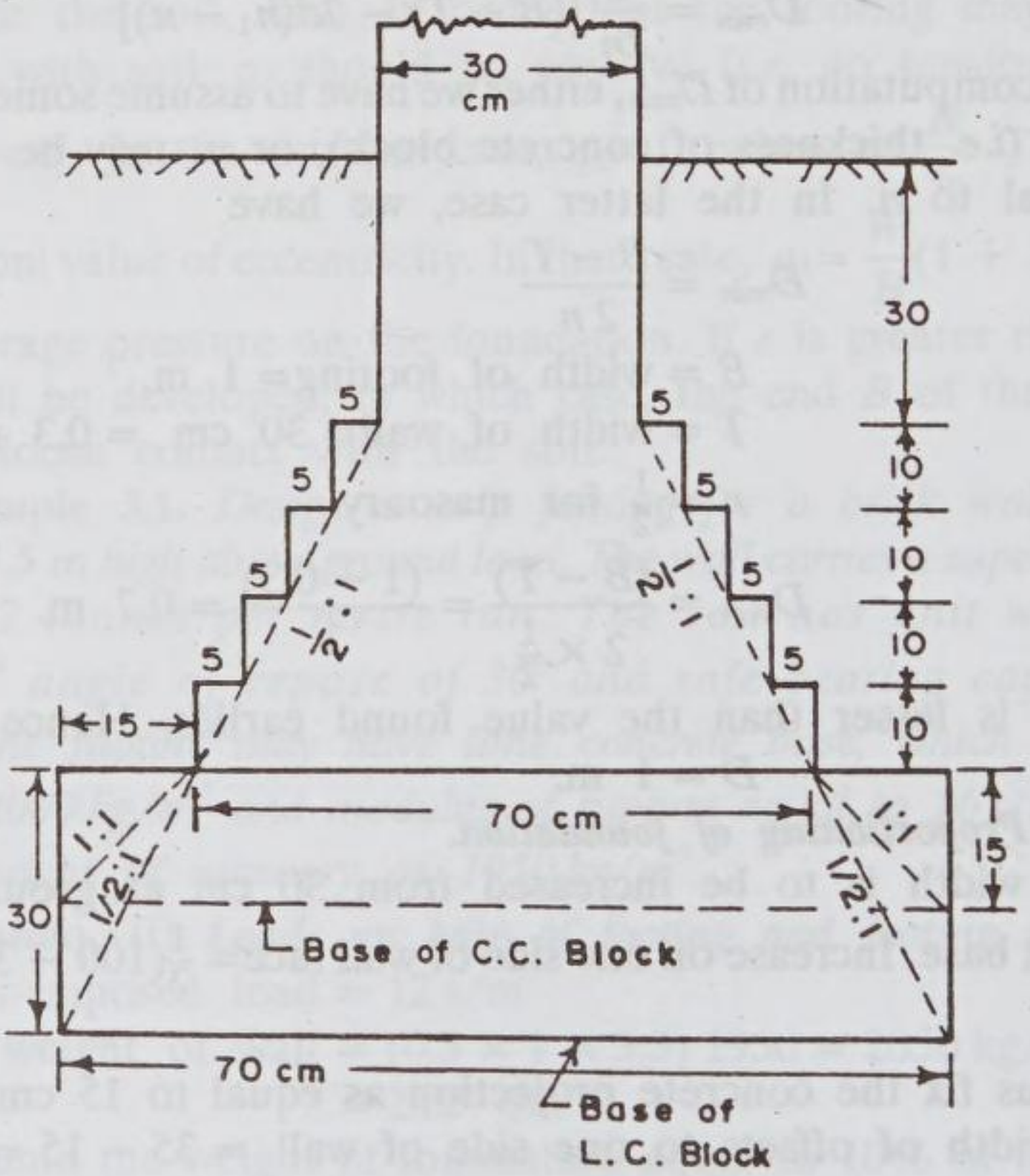


FIG. 3.8(*a*)

(*v*) *Check.*

Weight of foundation below ground level

$$= [(0.3 \times 0.3) + (0.4 \times 0.1) + (0.5 \times 0.1) + (0.6 \times 0.1) + (0.7 \times 0.1)]\ 1950 + [0.3 \times 1.0 \times 2000]$$

$$= 1250 \text{ kg/m}$$

$$= 1.205 \text{ t/m}$$

Actual assumed = 1.4 t/m

Hence safe.

Example 3.2. *Design the foundation for a stone pillar, 30 cm × 40 cm, carrying a super-imposed load of 300 kN at its top. The height of the pillar above ground level is 4 m. Take the unit weight of stone masonry as 22.5 kN/m³, and that of lean cement concrete*

as 23 kN/m². The soil has angle of repose of 25°, unit weight of 18 kN/m³ and safe bearing capacity of 150 kN/m². The foundation concrete may be in 1 : 4 : 8 cement concrete having safe modulus of rupture equal to 245 kN/m².

Solution. (*i*) ***Loads on base of footing, and footing dimensions.***

Super-imposed load = 300 kN

Self weight of pilla r = $0.3 \times 0.4 \times 4 \times 22.5 = 10.8$ kN.

Total $= 300 + 10.8 = 310.8.$

Assume weight of foundations @ 10% of the above $= 31.08$

∴ Total load on the base $= 310.8 + 31.08 = 342$ kN

∴ Base area required $= \frac{W}{q} = \frac{342}{150} = 2.28$ sq.m.

In order to have equal projections on the concrete block, and also to have uniform offsets all round, the dimensions B_1 and B_2 of the footing base will differ by the same magnitude by which the two sides of the column differ. Thus,

$$B_2 = (B_1 - 0.1) \text{ metres}$$

Also, $B_1 \times B_2 = A = 2.28 \text{ m}^2$

$$B_1 (B_1 - 0.1) = 2.28$$

or $B_1^2 - 0.1 B_1 - 2.28 = 0$

which gives $B_1 = 1.56$ m

Hence $B_2 = 1.56 - 0.1 = 1.46$ m

Total area $= 1.56 \times 1.46 = 2.278 \approx 2.28 \text{ m}^2$

Actual soil pressure $= \frac{342}{2.278} \approx 150 \text{ kN/m}^2$

(*ii*) ***Depth of footing.***

The minimum depth of footing, from Rankine's formula is

$$D_{min} = \frac{q}{\gamma}\left(\frac{1 - \sin\phi}{1 + \sin\phi}\right)^2$$

$$= \frac{150}{18}\left[\frac{1 - \sin 25°}{1 + \sin 25°}\right]^2 = 1.36 \text{ m.}$$

However, the minimum depth is also governed by Eq. 3.3(*a*) which is based on the requirement that the angle of spread of load should not exceed the permissible value.

∴ $$D_{min} = \frac{1}{2n}\left[(B_1 - b_1) - 2d\,(n_1 - n)\right]$$

For the computation of D_{min}, either we have to assume some

suitable value of the thickness d of concrete block, or n_1 may be assumed to be equal to n. In the latter case, we have

$$D_{min} = \frac{B_1 - b_1}{2n} = \frac{1.46 - 0.40}{2 \times \frac{1}{2}} = 1.06 \text{ m.}$$

This is less than the one found above.

Hence keep $D_{min} = 1.36$ m.

(iii) Proportioning of offsets.

The width is to be increased from 40 cm at ground level to 146 cm at the base. Increase in one side of column face $= \frac{1}{2}[146-40] = 53$ cm. Let us provide five offsets, each of value 7.5 cm. Hence the remaining value of concrete projection $= 53-37.5 = 15.5$ cm. Similarly, in the other direction, concrete projection $= \frac{1}{2}(136-30)-5\times7.5 = 15.5$ cm. Keep height of each offset equal to $7.5/\frac{1}{2} = 15$ cm.

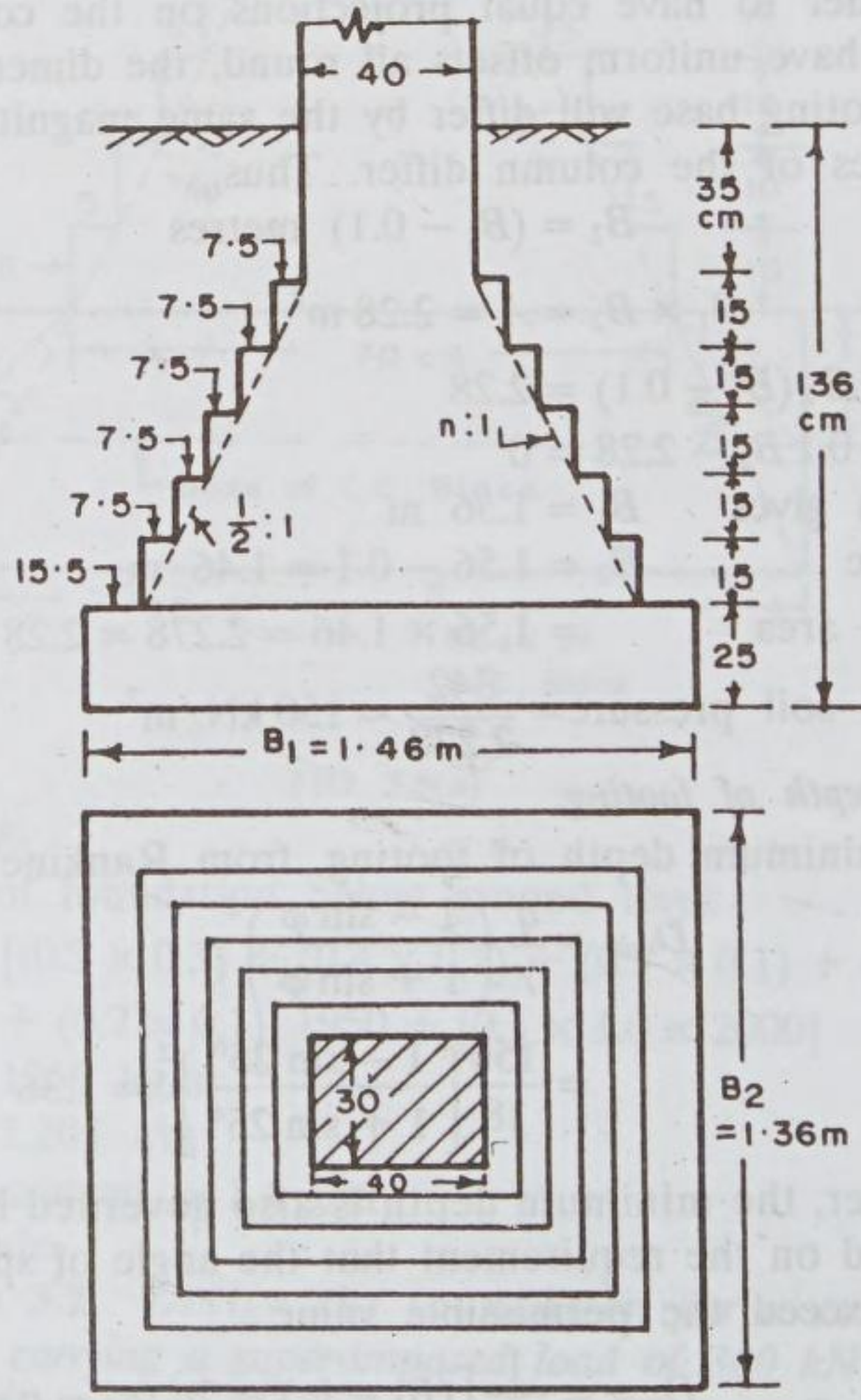


FIG. 3.8(*b*)

(iv) Thickness of concrete block.

Offset $a = 15.5$ cm.

Use 1 : 4 : 8 cement concrete having safe modulus of rupture equal to 245 kN/m^2.

$$\therefore \quad d = a\sqrt{\frac{3q}{m}}$$

$$= 15.5\sqrt{\frac{3 \times 150}{245}} = 21 \text{ cm..}$$

Keep this equal to 25 cm.

(iv) Check.

Weight of foundation below ground level

$$= [(0.4\times0.3\times0.36) + (0.55 \times 0.4\times0.15) + (0.7\times0.6\times0.15) + (0.85\times0.75\times0.15) + (1.0\times0.9\times0.15) + (1.15\times1.05\times0.15)] \times 22.5 + 1.46 \times 1.36 \times 0.25) \times 23$$

$$= 12.49 + 11.4 = 23.98 \text{ kN.}$$

This is less than the assumed weight = 31.08 kN of the foundation. Hence safe.

Example 3.3. *A residential building of three storey height has 30 cm thick brick wall of 12.5 m total height above ground level, inclusive of parapet wall. The wall supports two floor slabs and one roof slab, each having a clear span of 4 metres. The floor slabs are each 10 cm thick with 4 cm thick flooring. The roof slab is 12 cm thick with 13 cm terracing over it. The roof is accessible.*

Design the strip footing for the wall for the following data:

(i) Safe bearing pressure for the soil	*100 kN/m^2*
(ii) Angle of repose for the soil	*30°*
(iii) Unit weight of foundation soil	*18 kN/m^3*
(iv) Unit weight of concrete and flooring	*24 kN/m^3*
(v) Unit weight of lime concrete for foundation, and that of terracing	*20 kN/m^3*
(vi) Unit weight of brick masonry	*19.5 kN/m^3*
(vii) Safe modulus of rupture for lime concrete	*= 160 kN/m^2.*

Solution. (1) ***Load on the wall base at G.L. per metre run***

(i) Weight of wall = (12.5 × 0.3 × 1) 19.5 = 73.13 kN/m

(*ii*) *Weight transferred from roofing*

Dead load of R.C.C. slab, per m^2 area

$$= 0.12 \times 1 \times 1 \times 24$$

$$= 2.88\ kN/m^2$$

Dead load of roof finish

$$= 0.13 \times 1 \times 1 \times 20 = 2.60\ kN/m^2$$

Live load for accessible roof

$$= 1.5\ kN/m^2$$

(see Table 1.3 Chapter 1)

Total load $= 2.88 + 2.6 + 1.5 = 6.98\ kN/m^2$.

Since the span of roof slab is 4 m, half the load on this span will be transferred to this wall. Hence load per running metre of the wall

$$= \tfrac{1}{2}(4 \times 1 \times 6.98) = 13.96\ kN/m$$

(*iii*) *Weight transferred from two floors.*

The wall supports two floors. From Chapter 1, the load on the floors of residential buildings is $2\ kN/m^2$ (Table 1.2). However, the National Building Code of India (SP:7-1983) recommends that suitable reduction (see page 12, chapter 1) be made in live load on floors, if the number of floors exceed one. In the present case, the number of floors above the foundation *i.e.* design member under consideration, is *two* and the corresponding reduction in total live load on these two floors is 10%.

Hence design live load on each floor

$$= 2 \times 0.9 = 1.8\ kN/m^2$$

Dead load, per square metre of the floor, including floor finish

$$= (0.10 + 0.04) \times 1 \times 1 \times 24 = 3.36\ kN/m^2$$

Dead load + live load on each floor $= 1.8 + 3.36$

$$= 5.16\ kN/m^2$$

Each floor of span 4 m transfers half the above load to the wall under consideration. Since there are *two* floors, load transferred to the wall per metre run

$$= 2\,(\tfrac{1}{2} \times 4 \times 1 \times 5.16) = 20.64\ kN/m$$

(*iv*) *Total load at ground level*

$$= 73.13 + 13.96 + 20.64 = 107.70\ kN/m$$

2. *Width of footing.* Let us assume the weight of footing @10% of the load transferred to the wall at ground level.

$\therefore$ Total load at footing base $= 1.1 \times 107.73 = 118.5\ kN/m$

$$\text{Footing width} = \frac{118.5}{100} = 1.185\ m.$$

Provide $B = 1.2$ m.

3. *Depth of footing.* Minimum depth of footing, as given by Rankine's formula is

$$D_{min} = \frac{q}{\gamma}\left(\frac{1-\sin\phi}{1+\sin\phi}\right)^2$$

where q = soil pressure $= \frac{118.5}{1.2} = 98.75\ \text{kN/m}^2$

γ = unit weight of soil $= 18\ (\text{kN/m}^3)$,

$\phi = 30°$

$$\therefore \quad D_{min} = \frac{98.75}{18}\left(\frac{1-\sin 30°}{1+\sin 30°}\right)^2 = \frac{98.75}{18\times 9} = 0.6\ \text{m}$$

However, the minimum depth is also governed by Eq. 3.3 (*a*)

$$D_{min} = \frac{1}{2n}[(B-T) - 2d\,(n_1 - n)]$$

For lime concrete base $n_1 = \frac{2}{3}$.

For masonry, $n = \frac{1}{2}$

$$\therefore \quad D_{min} = \frac{1}{2\times\frac{1}{2}}\left[(B-T) - 2d\left(\frac{2}{3}-\frac{1}{2}\right)\right]$$

$$= (B-T) - \frac{d}{3}$$

Let us assume thickness of concrete block as 20 cm. Also, $B = 1.2$ m and $T = 0.3$ m.

$$\therefore \quad D_{min} = (1.2 - 0.3) - \frac{0.2}{3} = 0.83\ \text{m}$$

Thus, the minimum depth, greater of the two, will be 0.83 m. However, provide actual depth of footing equal to 0.9 m.

Note. It is essential to ensure through field investigation, than the safe bearing pressure of $100\ \text{kN/m}^2$ is available at this depth.

4. Proportioning of foundation. The width is to be increased from 30 cm at ground level to 120 cm at the base, the increase being equal to $\frac{1}{2}(120-30)=45$ cm on each side. Fixing the concrete projection equal to 15 cm, total width of offsets to one side $=45-15=30$ cm. Keeping 5 cm offsets, no. of offsets $= 30/5 = 6$. The minimum height of each offset $= 5/n = 5/y_2 = 10$ cm. Hence height of footing with offsets $= 6 \times 10 = 60$ cm.

5. Thickness of concrete block. Offset $a = 15$ cm

$$\therefore \quad d = a\sqrt{\frac{3q}{m}}$$

where q = bearing pressure on soil $= 98.75\ \text{kN/m}^2$

m = safe modulus of rupture for lime concrete $= 1[illegible]0\ \text{kN/m}^2$

$$\therefore \qquad d = 15\sqrt{\frac{3 \times 98.75}{160}} = 20.4 \text{ cm}$$

Minimum value of d with a dispersion of $n_1 : 1$ is

$$d_{min} = \frac{a}{n_1} = \frac{15}{2/3}$$

$$= \frac{15 \times 3}{2} = 22.5 \text{ cm.}$$

Hence keep $d = 22.5$ cm.

6. ***Check.*** The foundation section is shown in Fig. 3.9. Weight of foundation below G.L.

$$= [(0.3 \times 0.075) + (0.4 + 0.5 + 0.6 + 0.7 + 0.8 + 0.9) \times 0.1]\ 1.95 + [0.225 \times 1.2 \times 20]$$

$$= 8.04 + 5.4 = 13.44 \text{ kN/m}$$

(Against assumed value of $0.1 \times 107.73 = 10.77$ kN)

Super-imposed load

$$= 107.73$$

$\therefore$ Total load

$$= 13.44 + 107.73$$

$$= 121.17 \text{ kN}$$

$$\text{Soil pressure} = \frac{121.17}{1.2}$$

$$\approx 101 \text{ kN/m}^2$$

This is slightly more than safe bearing pressure of100 kN/m^2, the increase being only 1%. Hence O.K.

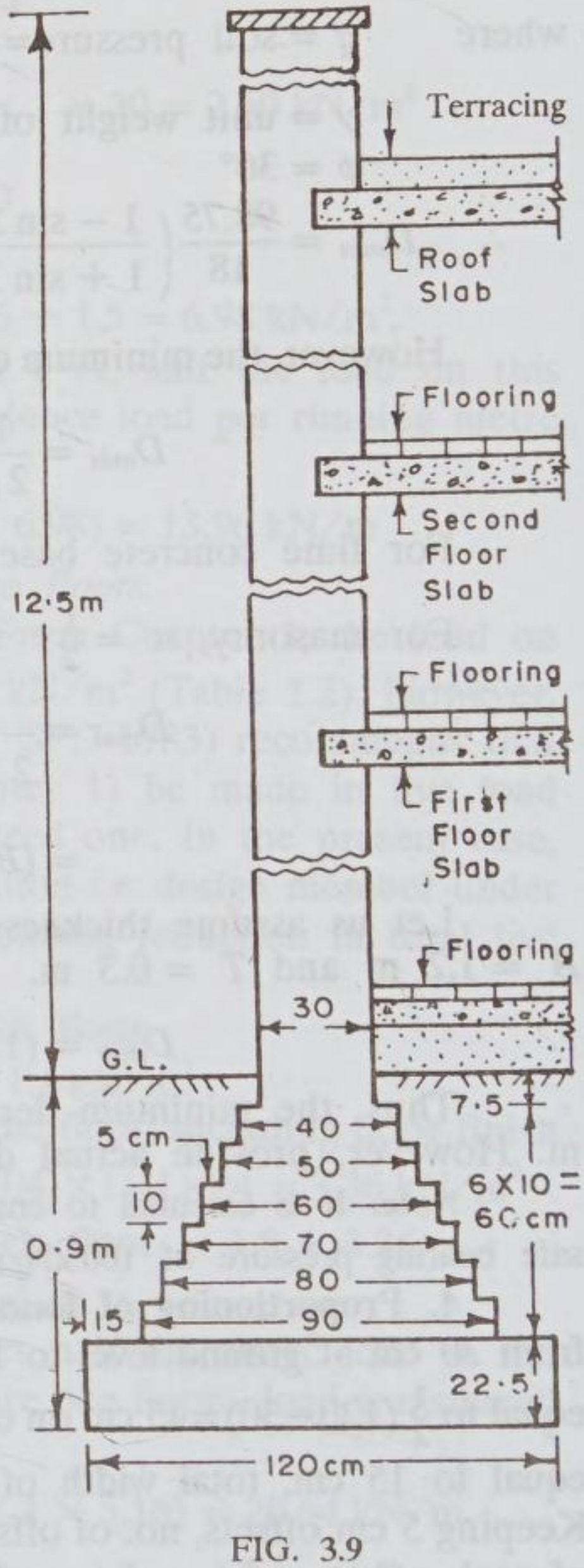


FIG. 3.9

3.6. GRILLAGE FOUNDATIONS

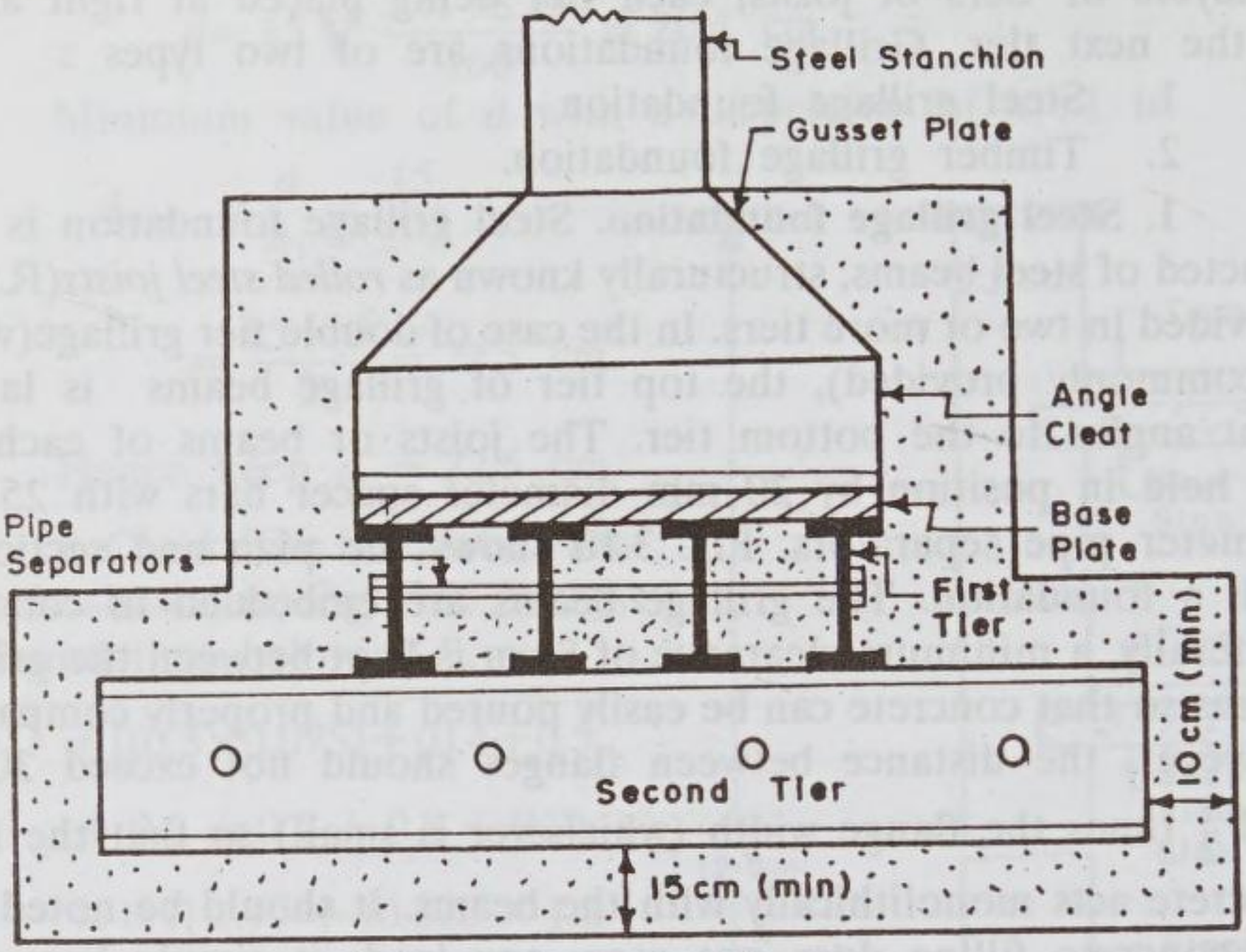
A *grillage foundation* is a special type of isolated footing, generally provided for heavily loaded steel stanchions, specially in those locations where bearing capacity of soil is poor. The depth of foundation is limited from 1 m to 1.5 m. The load of the column or stanchion is distributed or spread to a very large area by means

of layers or tiers of joists, each tier being placed at right angles to the next tier. Grillage foundations are of two types :

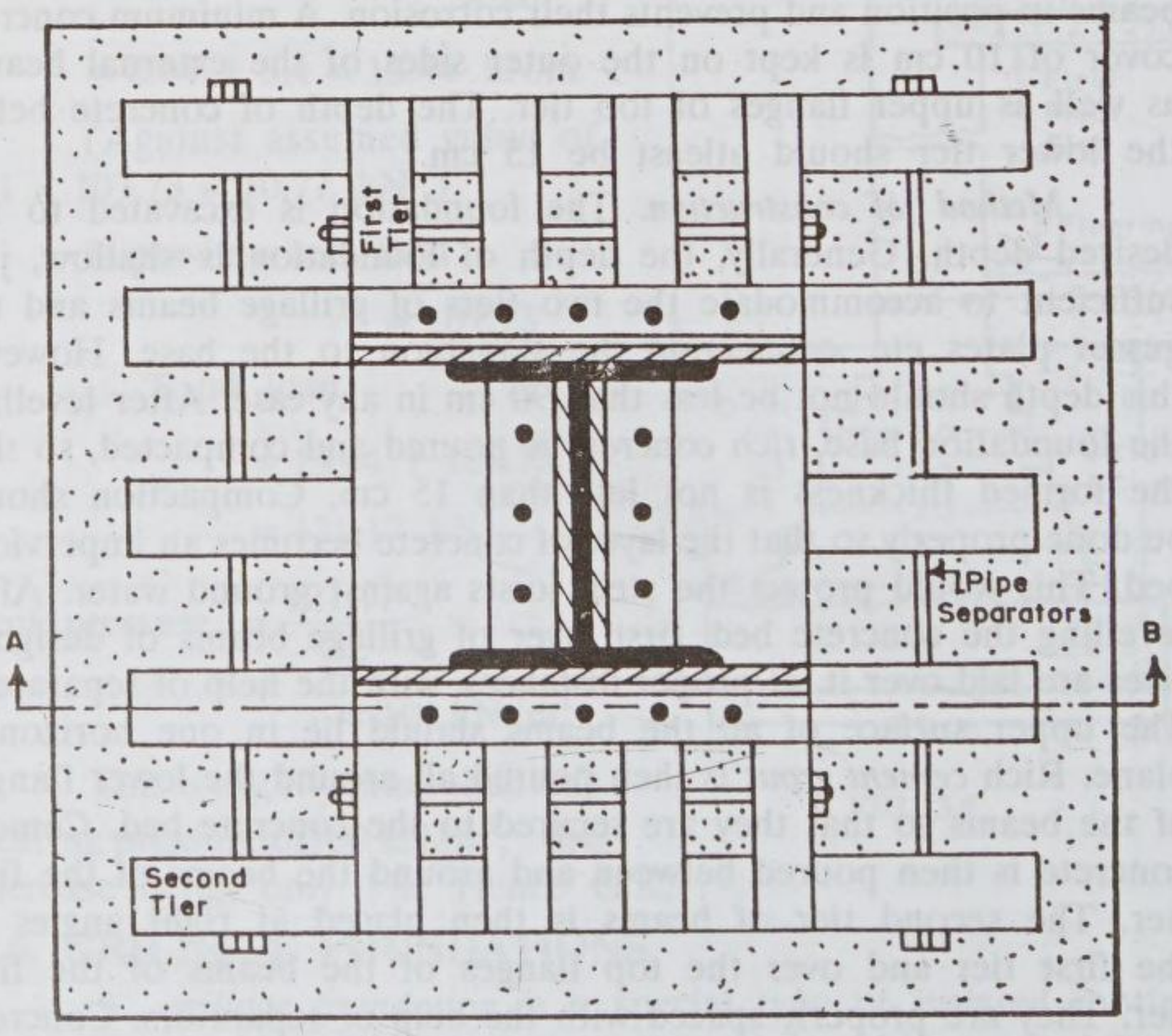
1. Steel grillage foundation
2. Timber grillage foundation.

1. **Steel grillage foundation.** Steel grillage foundation is constructed of steel beams, structurally known as *rolled steel joists*(R.S.J.), provided in two or more tiers. In the case of double tier grillage(which is commonly provided), the top tier of grillage beams is laid at right angles to the bottom tier. The joists or beams of each tier are held in position by 20 mm diameter spacer bars with 25 mm diameter pipe separators. Fig. 3.10 shows the plan and section of such a foundation. The grillage beams are embedded in concrete. Generally, a minimum clearance of 8 cm is kept between the grillage beams so that concrete can be easily poured and properly compacted. However, the distance between flanges should not exceed 30 cm or $1\frac{1}{2}$ times the flange width (whichever is small) so that the filled concrete acts monolithically with the beams. It should be noted that the concrete filling does not carry any load; it simply keeps the beams in position and prevents their corrosion. A minimum concrete cover of 10 cm is kept on the outer sides of the external beams, as well as upper flanges of top tier. The depth of concrete below the lower tier should atleast be 15 cm.

Method of construction. The foundation is excavated to the desired depth. Generally, the depth of foundation is shallow, just sufficient to accommodate the two tiers of grillage beams and the gusset plates etc. connecting the stanchion to the base. However this depth should not be less than 90 cm in any case. After levelling the foundation base, rich concrete is poured and compacted, so that the formed thickness is not less than 15 cm. Compaction should be done properly so that the layer of concrete becomes an impervious bed. This would protect the steel joists against ground water. After levelling the concrete bed, first layer of grillage beams of designed sizes are laid over it, at proper distances, with the help of separaters. The upper surface of all the beams should lie in one horizontal plane. Rich *cement grout* is then poured all around the lower flanges of the beams so that they are secured to the concrete bed. Cement concrete is then poured between and around the beams of the first tier. The second tier of beams is then placed at right angles to the first tier and over the top flanges of the beams of the first tier. They are properly spaced with the help of separators. Concrete is then poured between and around the steel beams. The steel stanchion is then connected to the upper tier with the help of a base plate, side angles and gusset plate. These connecting elements are also embedded in the concrete so that joint becomes rigid..



(a) Section at AB.



(b) Plan

FIG. 3.10. TYPICAL GRILLAGE FOUNDATION FOR STEEL STANCHION.

Steel grillage foundation may also be provided for a masonry wall on soils of low bearing capacity. The grillage foundation for such a case consists of only one tier, though in some circumstances when the wall is wider and it carries heavy loads, two tiers may also be provided. Fig. 3.11 shows the details for both the cases.

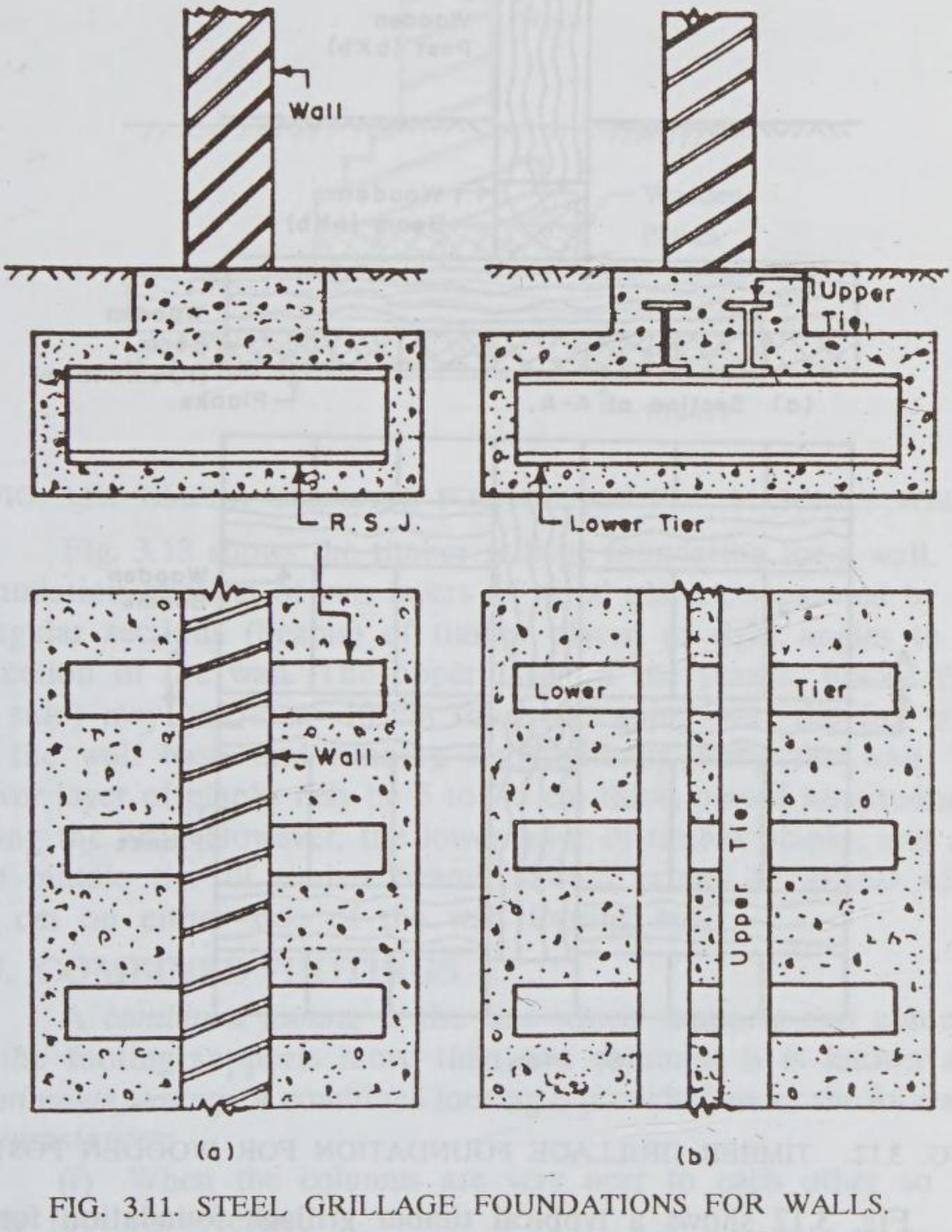


FIG. 3.11. STEEL GRILLAGE FOUNDATIONS FOR WALLS.

2. Timber grillage foundation. Timber grillage foundation is provided for heavily loaded timber column or masonry wall. The foundation uses timber planks and timber beams in the place of steel joists. This foundation is specially useful in water logged areas where the bearing power of the soil is very low, and where the steel beams may get corroded due to subsoil water. The loading on the soil is limited to 50 to 60 kN/m^2. No concrete is embedded between the timber joists. However, the bottom concrete (provided

in steel grillage foundation is replaced by timber platform constructed of timber planks.

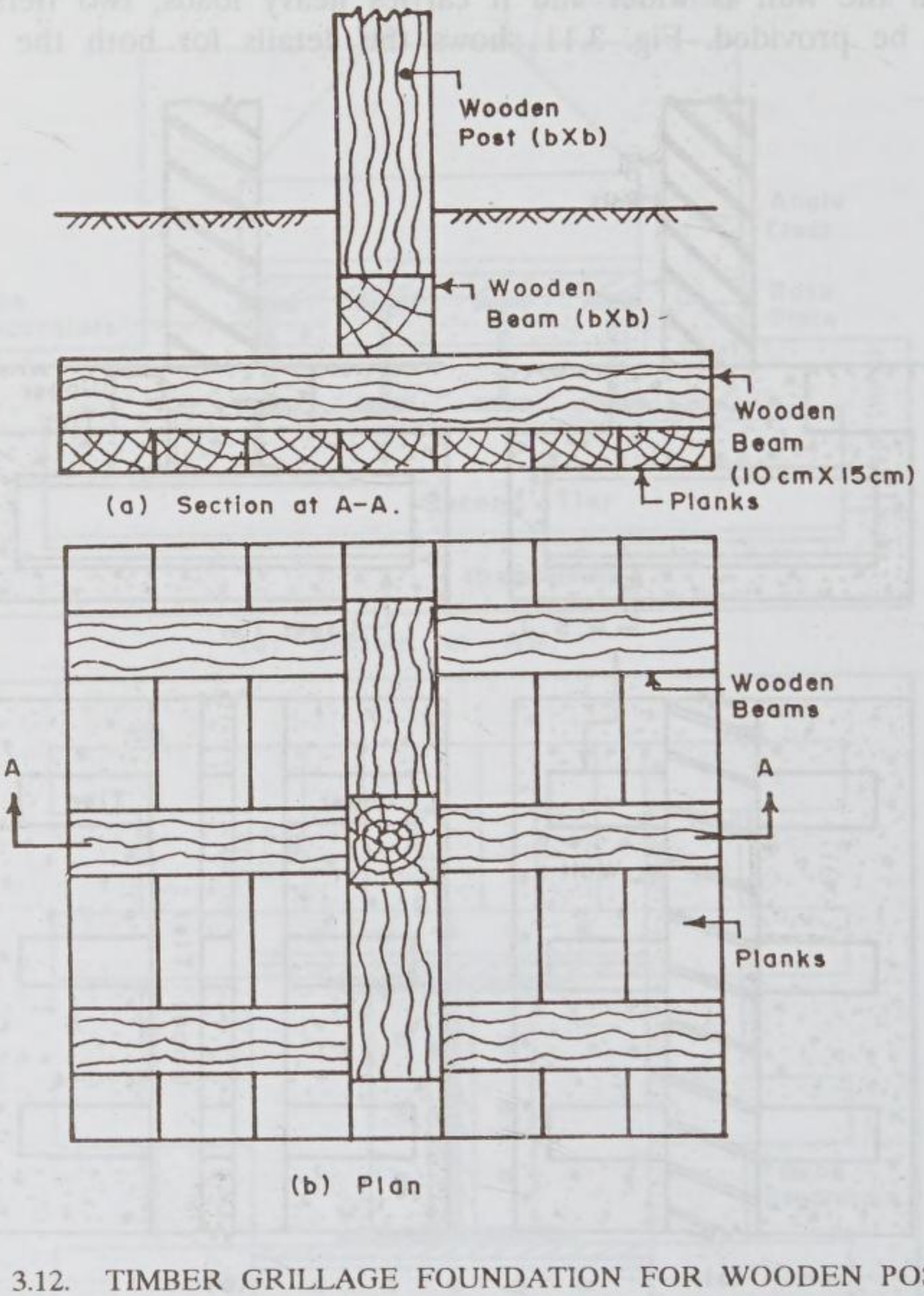


FIG. 3.12. TIMBER GRILLAGE FOUNDATION FOR WOODEN POST.

Fig. 3.12 shows a typpical timber grillage foundation for a timber column. After excavating the foundation of the desired depth and levelling it, the bottom layer of planks 5 to 7.5 cm thick and 20 to 30 cm wide is laid. The planks are arranged side by side, without any gap between them. Over this platform, a tier of wooden beams, about 15 cm × 10 cm in size, spaced 30 to 50 cm apart, is laid at right angles to the direction of the planks. Over the top of this layer, a timber beam of the same section as that of the

wood post is placed at right angles. The timber post is then fixed at right angles to this timber beam.

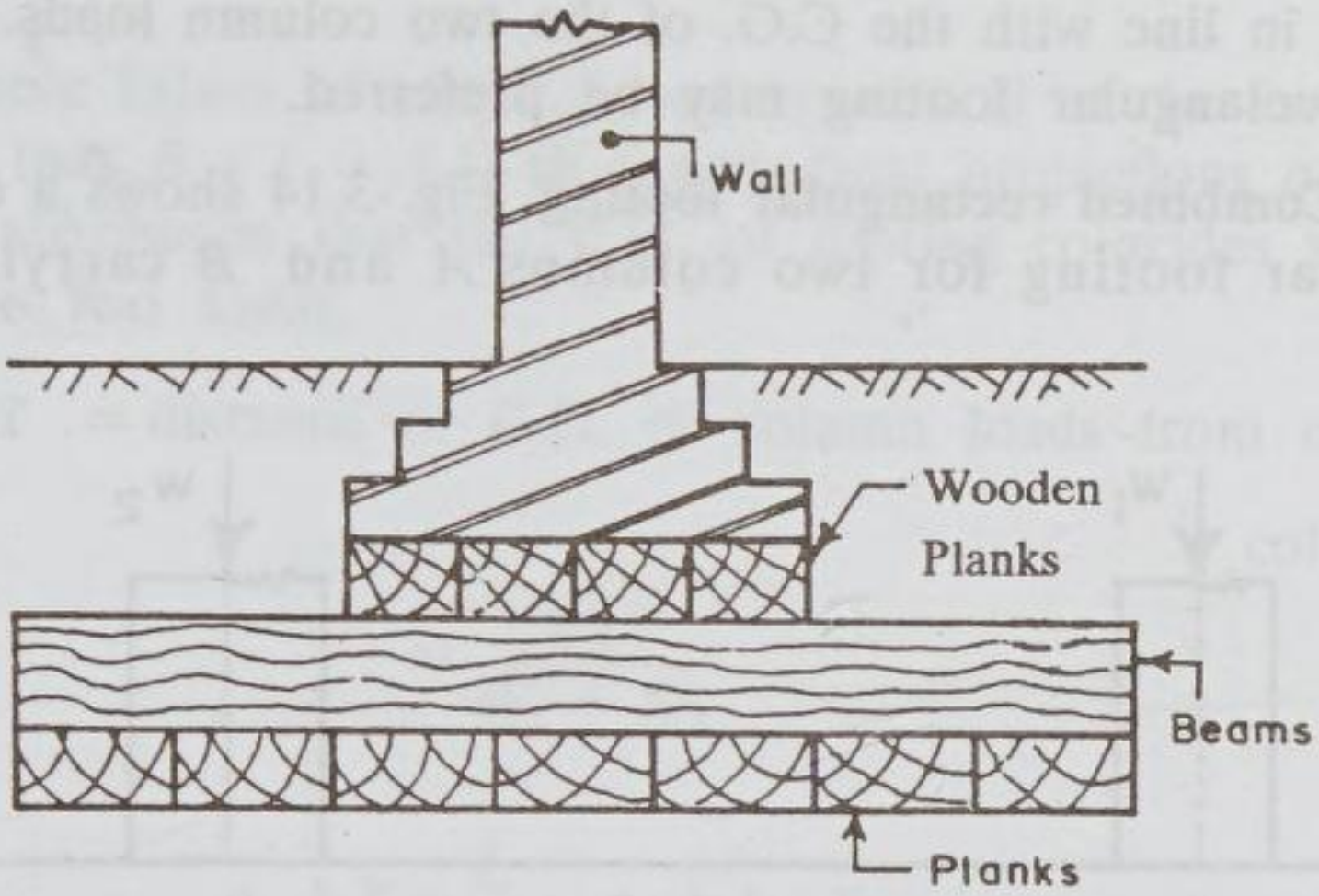


FIG. 3.13. TIMBER GRILLAGE FOUNDATION FOR MASONRY WALL.

Fig. 3.13 shows the timber grillage foundation for a wall. The foundation consists of two layers of wood planks, separated by rectangular sections (beams) of timber placed at right angles to the direction of the wall. The upper layer of the planks, placed (side by side) may be 7.5 to 10 cm thick, extending over the full width of the wall base, and running longitudinally along the wall. The lower layer of planks may be 5 to 7.5 cm thick, placed longitudinally along the wall. However, the lower layer of timber planks, and also the middle tier of timber beams, should extend to atleast 45 to 60 cm on either side of the wall footing base.

3.7. COMBINED FOOTINGS

A *combined footing* is the one which supports two columns. If the footing supports more than two columns, it is known as a *continuous footing.* A combined footing is provided under the following circumstances:

(*i*) When the columns are very near to each other so that their footings overlap.

(*ii*) When the bearing capacity of the soil is less, requiring more area under individual footing.

(*iii*) When the end column is near a property line so that its footing cannot spread in that direction.

A combined footing may be rectangular or trapezoidal in plan. The aim is to get uniform pressure distribution under the footing. For this, the centre of gravity (C.G.) of the footing area should

coincide with the C.G. of the combined loads of the two columns. If the outer column, near the property line, carries heavier load, provision of trapezoidal column becomes essential to bring the C.G. of footing in line with the C.G. of the two column loads. In other cases, a rectangular footing may be preferred.

1. Combined rectangular footing. Fig. 3.14 shows a combined rectangular footing for two columns *A* and *B* carrying loads

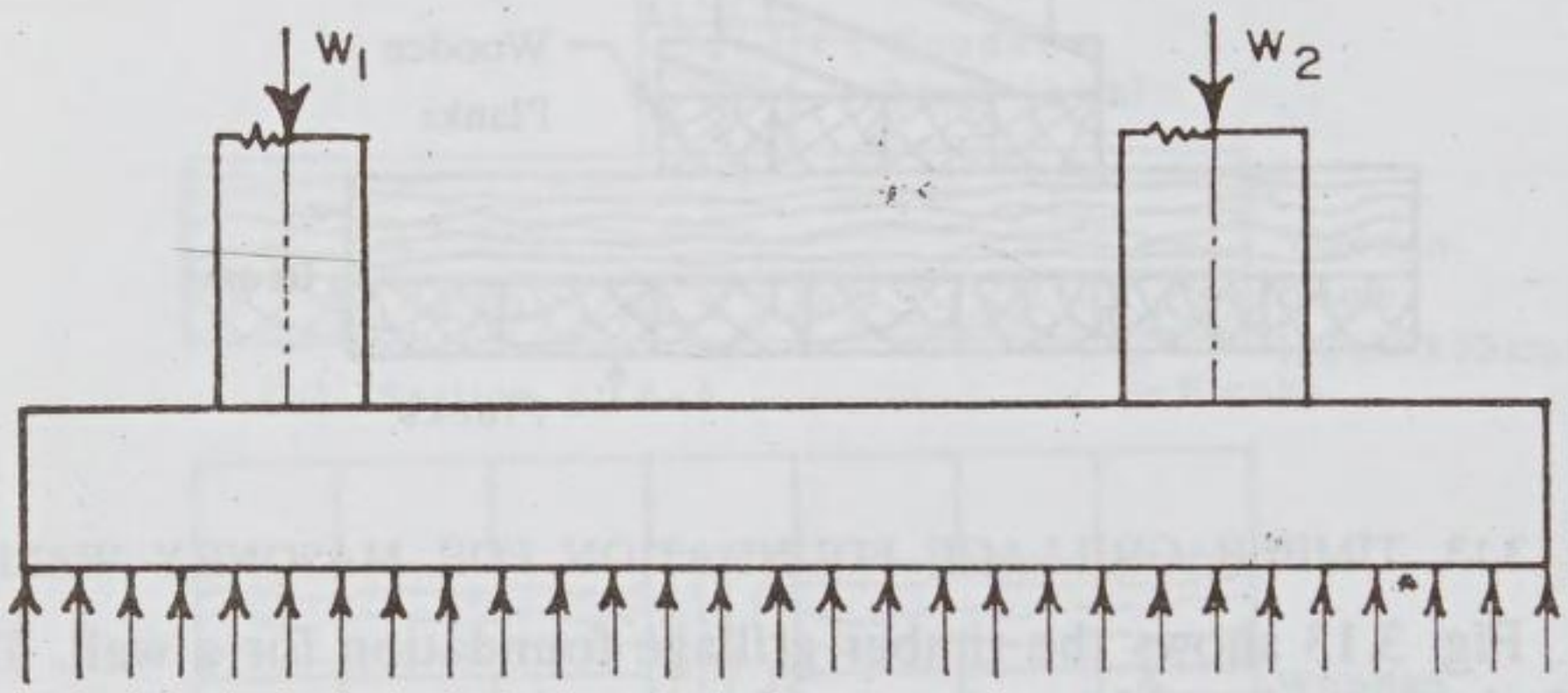


FIG. 3.14. COMBINED RECTANGULAR FOOTING.

W_1 and W_2 , and spaced l centre to centre. If W' is the weight of the footing, and q_s is the safe bearing capacity, the footing area is given by

$$A = \frac{W_1 + W_2 + W'}{q_s} \qquad ...(3.8)$$

Suitable values of length L and breadth B of the footing are chosen, so that $B \times L = A$. The longitudinal projections a_1 and a_2 should be so chosen that the C.G. of footing coincides with the C.G. of the two loads.

Let $\bar{x}$ = distance of C.G. of column loads from centre of column A

$$= \frac{W_2 l}{W_1 + W_2} \qquad ...(3.9)$$

Then $$a_1 + \bar{x} = \frac{L}{2} = a_2 + l - \bar{x} \qquad ...(3.10)$$

From the above, the projections a_1 and a_2 can be determined. The net upward pressure p_o is given by

$$p_0 = \frac{W_1 + W_2}{L \, . \, B}$$

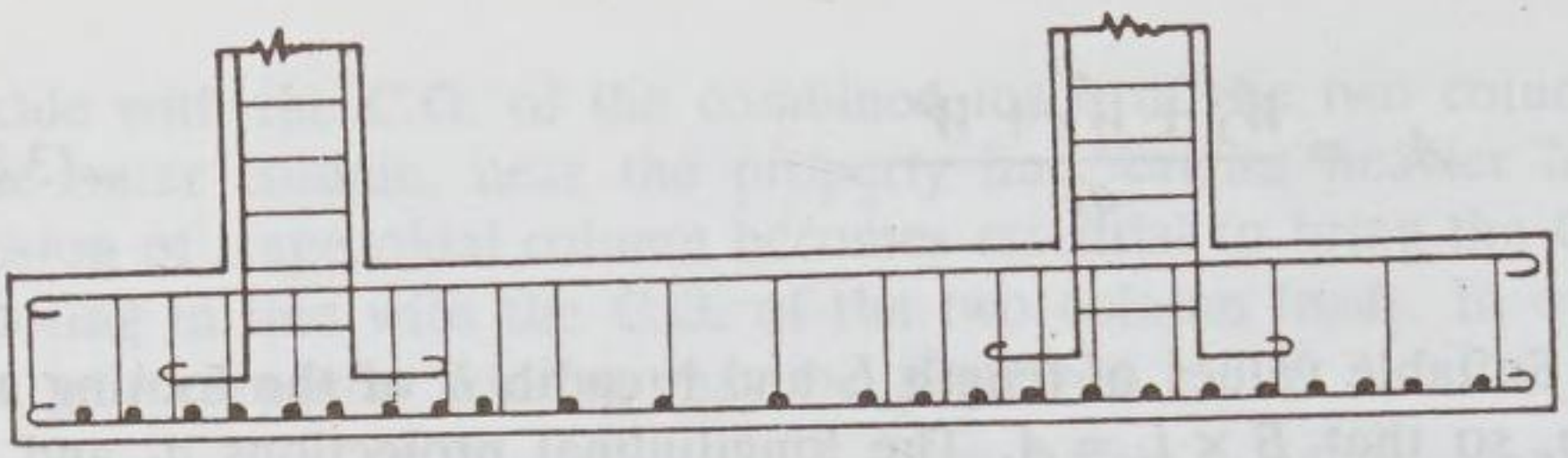
This net pressure intensity is used for structural analysis and design of combined footing. A *combined foundation* may be either of reinforced cement concrete (R.C.C.) or of steel grillage type.

(*i*) ***Combined rectangular footing of R.C.C.*** A rectangular footing of R.C.C. consists of a reinforced concrete slab which is designed for both longitudinal bending as well as transverse bending. If the distance between the columns is large, a longitudinal beam may be provided, joining columns. Typical details of a R.C. footing, without longitudinal beam, are shown in Fig. 3.15. Fig. 3.16 shows typical details of rectangular footing, having longitudinal beam. The longitudinal beam may be provided either below the footing slab, or it may project above the slab.

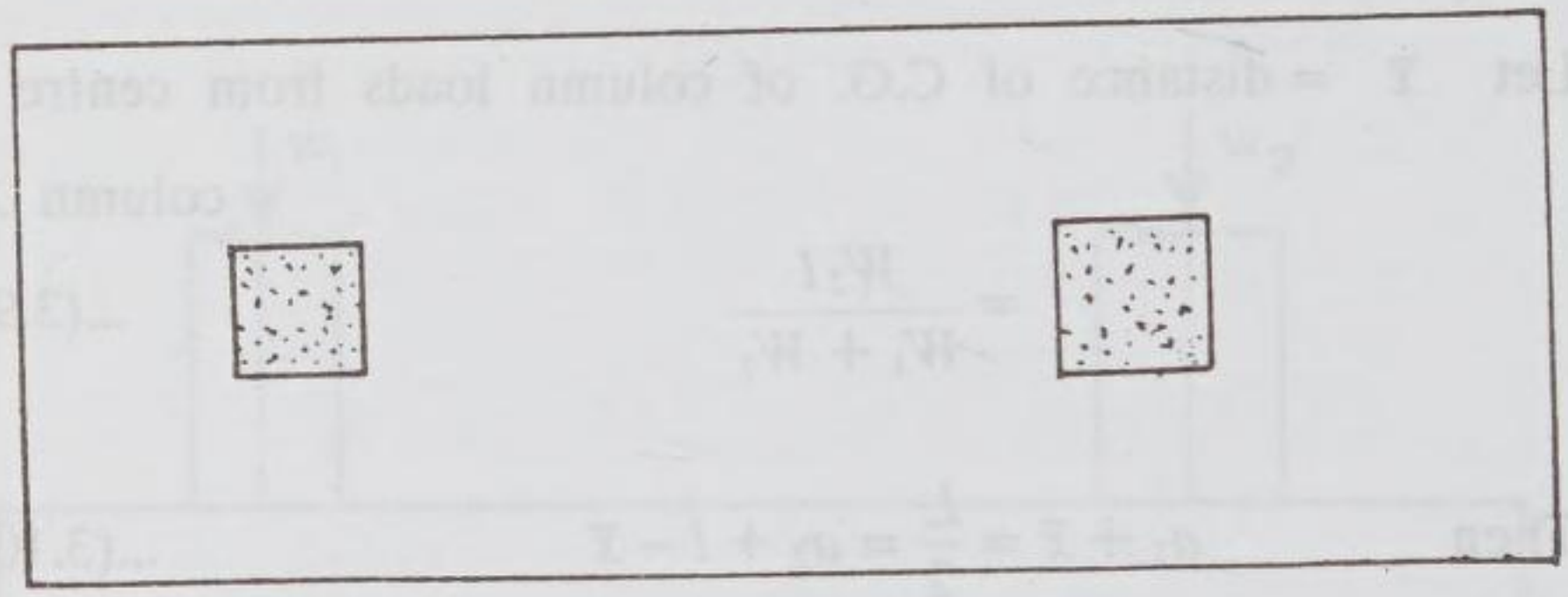
(*ii*) ***Combined steel grillage rectangular footing.*** Such a footing is provided to support two steel stanchions. The upper tier of steel joists receives the loads from the two columns and transfers the load to the lower tier. Fig. 3.17 shows typical details.

2. Combined trapezoidal footing. When two column loads are unequal, with the outer column carrying heavier load and when there is space limitation beyond the outer column, a trapezoidal combined footing is provided

Fig. 3.18 shows two columns, with load W_1 and W_2, spaced at distance l apart. Load W_1 is greater than W_2 . Let L be the

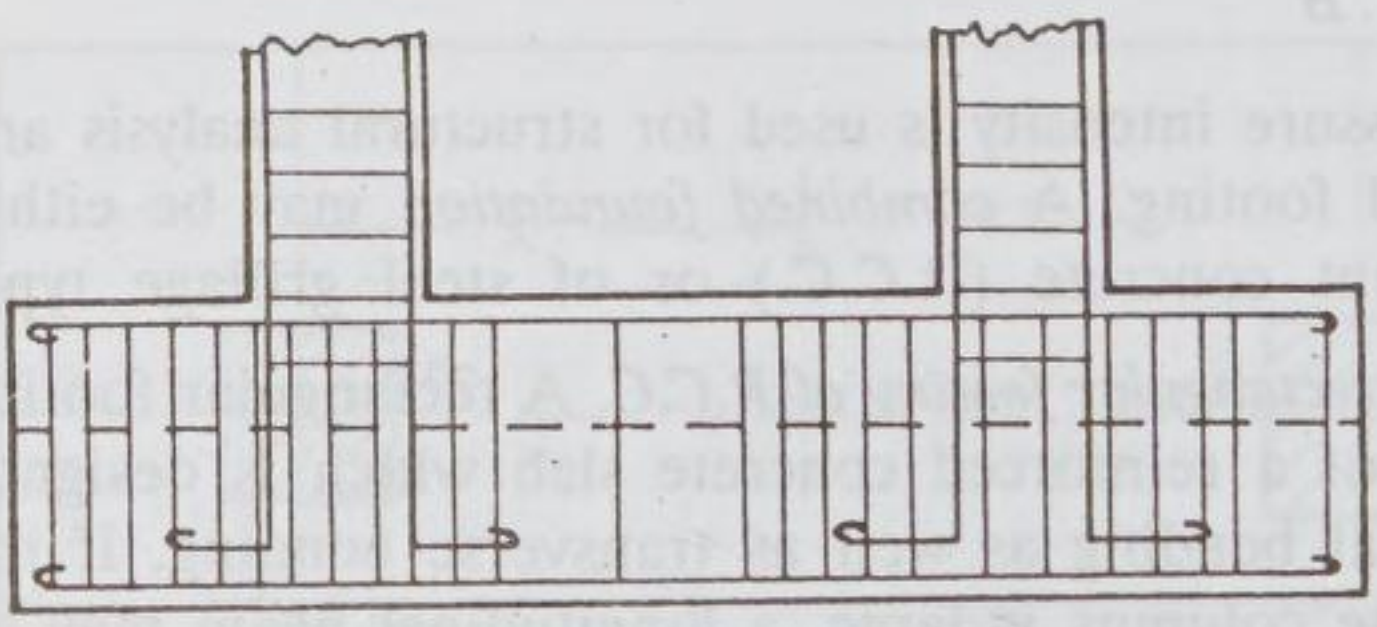


(a) Longitudinal Section

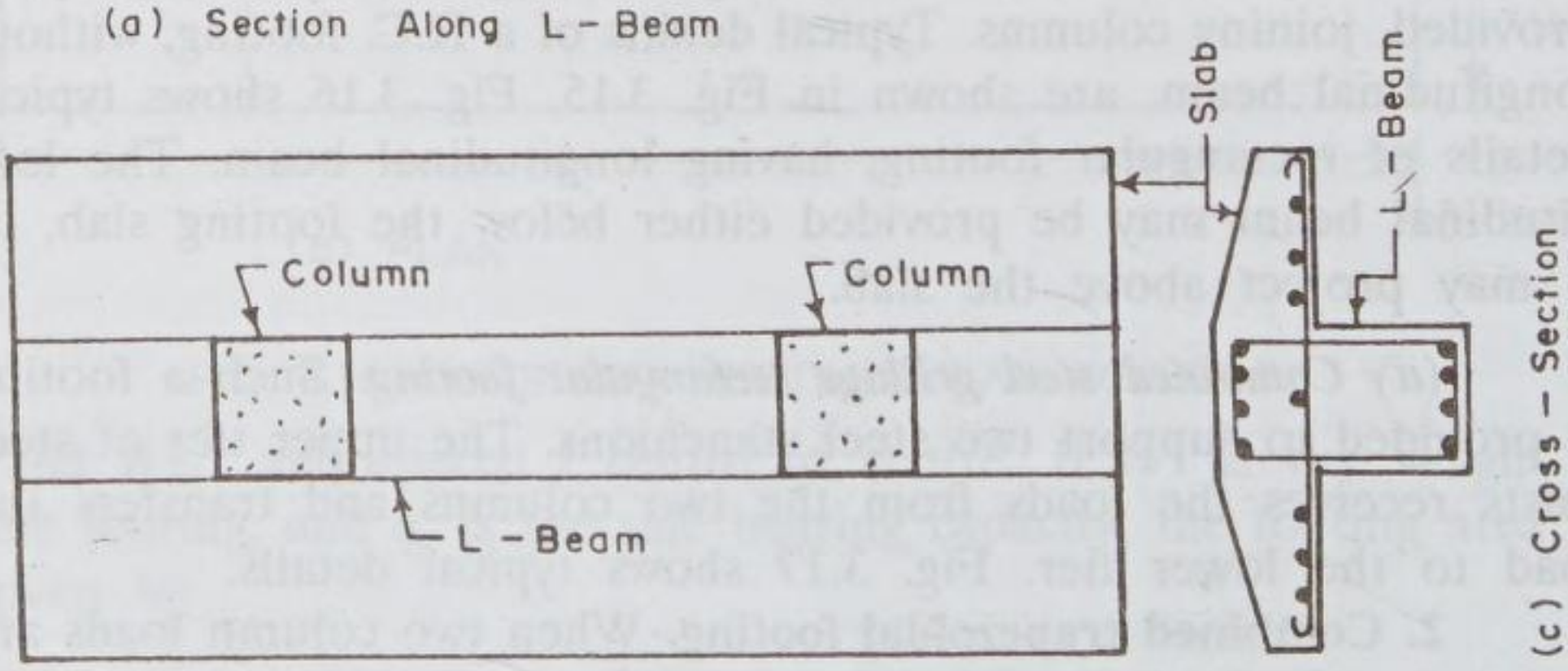


(b) Top Plan

FIG. 3.15. RECTANGULAR R.C.C. FOOTING WITHOUT LONGITUDINAL BEAM.



(a) Section Along L - Beam



(b) Top Plan

FIG. 3.16. RECTANGULAR R.C.C. FOOTING WITH LONGITUDINAL BEAM.

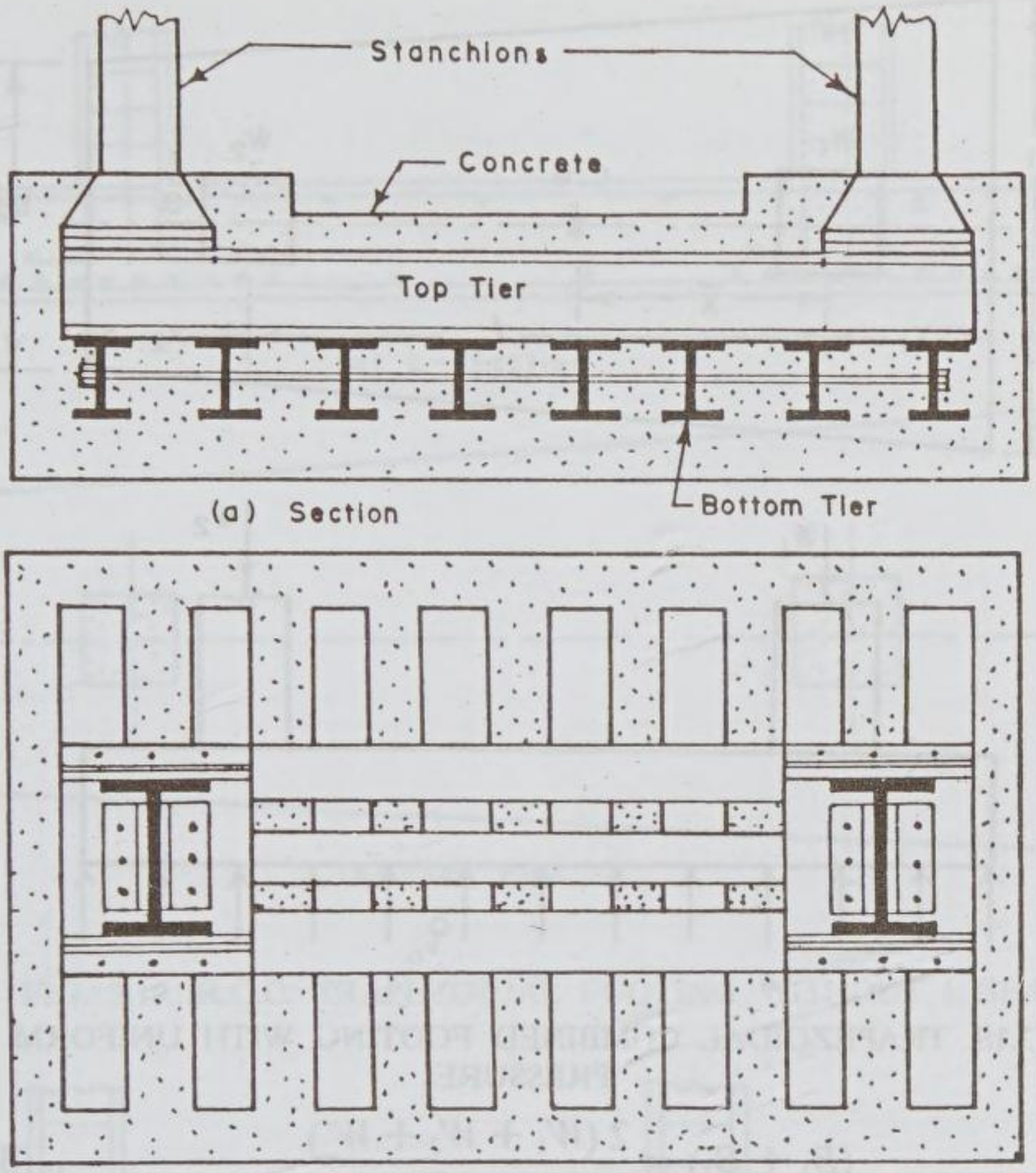


FIG. 3.17. COMBINED STEEL GRILLAGE RECTANGULAR FOOTING.

length of the footing and a_1 and a_2 be the cantilever projections which can be suitably fixed. The widths B_1 and B_2 are unknowns.

$$\text{Area of footing} = \frac{B_1 + B_2}{2} L \qquad ...(i)$$

If W' is the weight of footing and q_s is the safe bearing capacity,

$$\text{Area required.} = \frac{W_1 + W_2 + W'}{q_s} \qquad ...(ii)$$

Equating the two,

$$(B_1 + B_2)\frac{L}{2} = \frac{W_1 + W_2 + W'}{q_s}$$

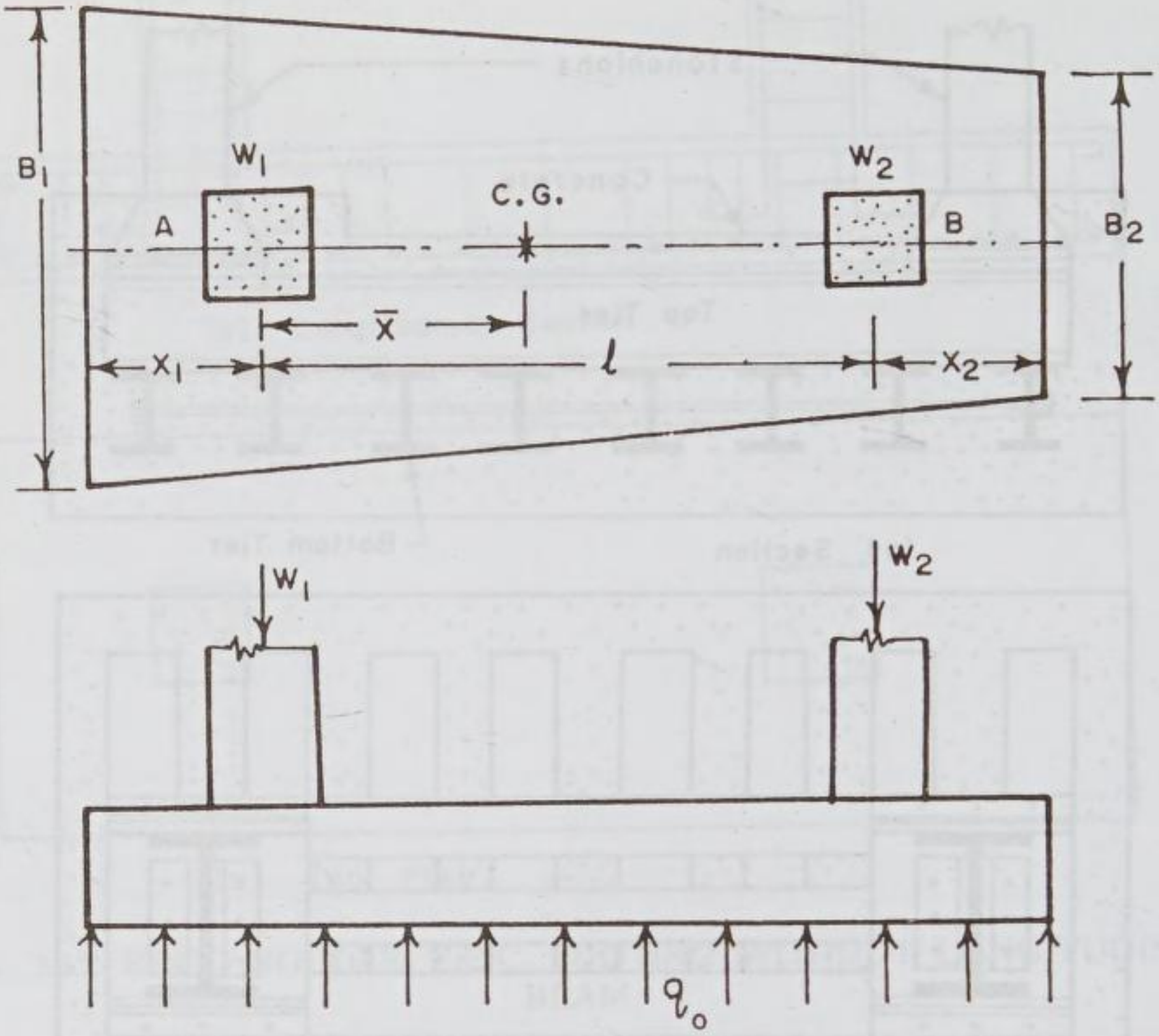


FIG. 3.18. TRAPEZOIDAL COMBINED FOOTING WITH UNIFORM SOIL PRESSURE.

or $$(B_1 + B_2) = \frac{2\,(W_1 + W_2 + W')}{q_s \,.\, L} \qquad ...(3.11)$$

Also distance of C.G. of load from W_1 is given by

$$\bar{x} = \frac{W_2 l}{W_1 + W_2} \qquad ...(iii)$$

Distance of C.G. of trapezium from long edge

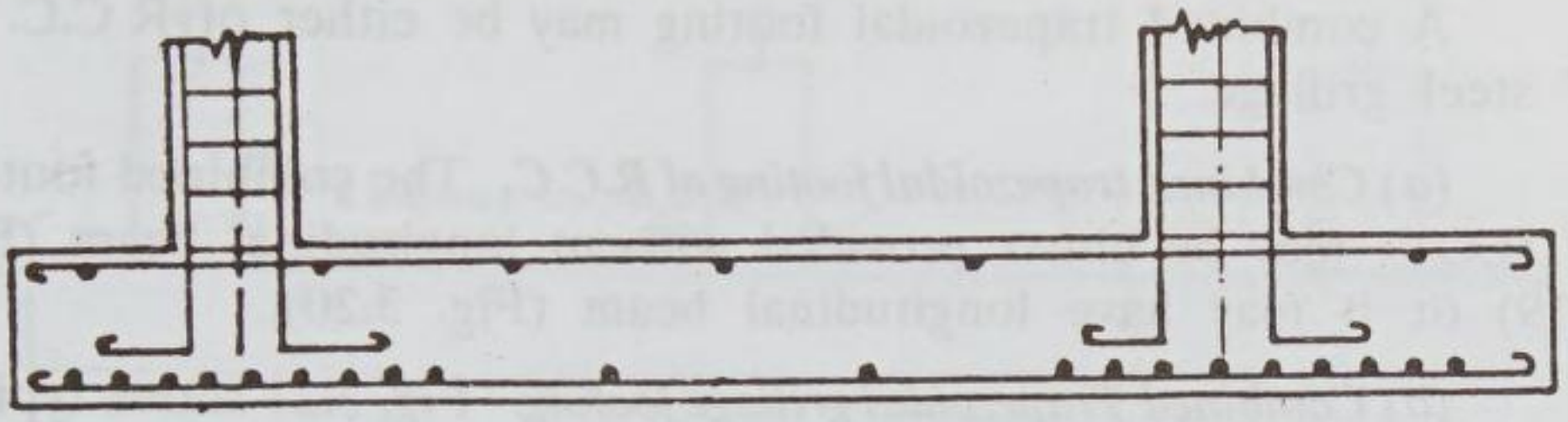
$$= \frac{L}{3}\left(\frac{B_1 + 2B_2}{B_1 + B_2}\right) \qquad ...(iv)$$

From (*iii*) and (*iv*) we have

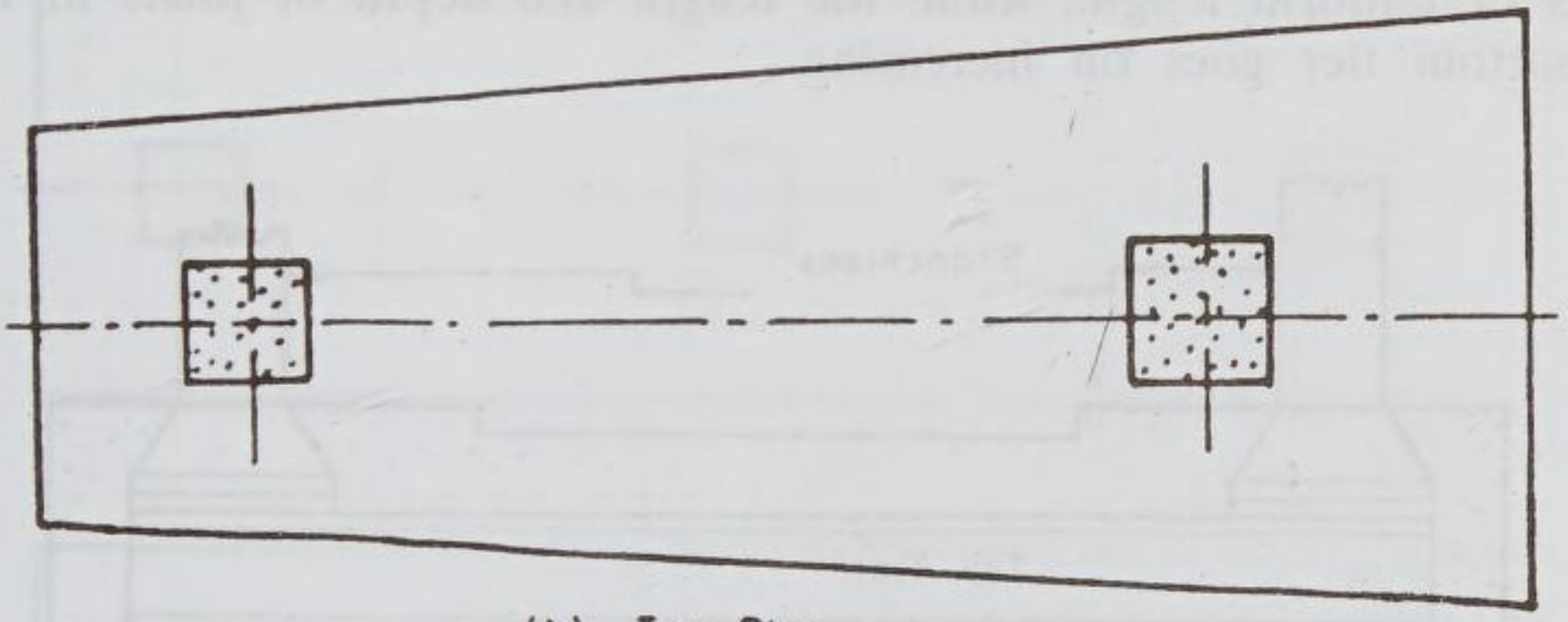
$$a_1 + \frac{W_2 l}{W_1 + W_2} = \frac{L}{3}\left(\frac{B_1 + 2B_2}{B_1 + B_2}\right) \qquad ...(3.12)$$

From Eqs. 3.11 and 3.12, unknowns B_1 and B_2 can be determined. The *net* upward soil pressure intensity p_o will be *uniform* throughout, and its magnitude is given by

$$p_o = \frac{W_1 + W_2}{\frac{1}{2}(B_1 + B_2) L} \qquad ...(3.13)$$

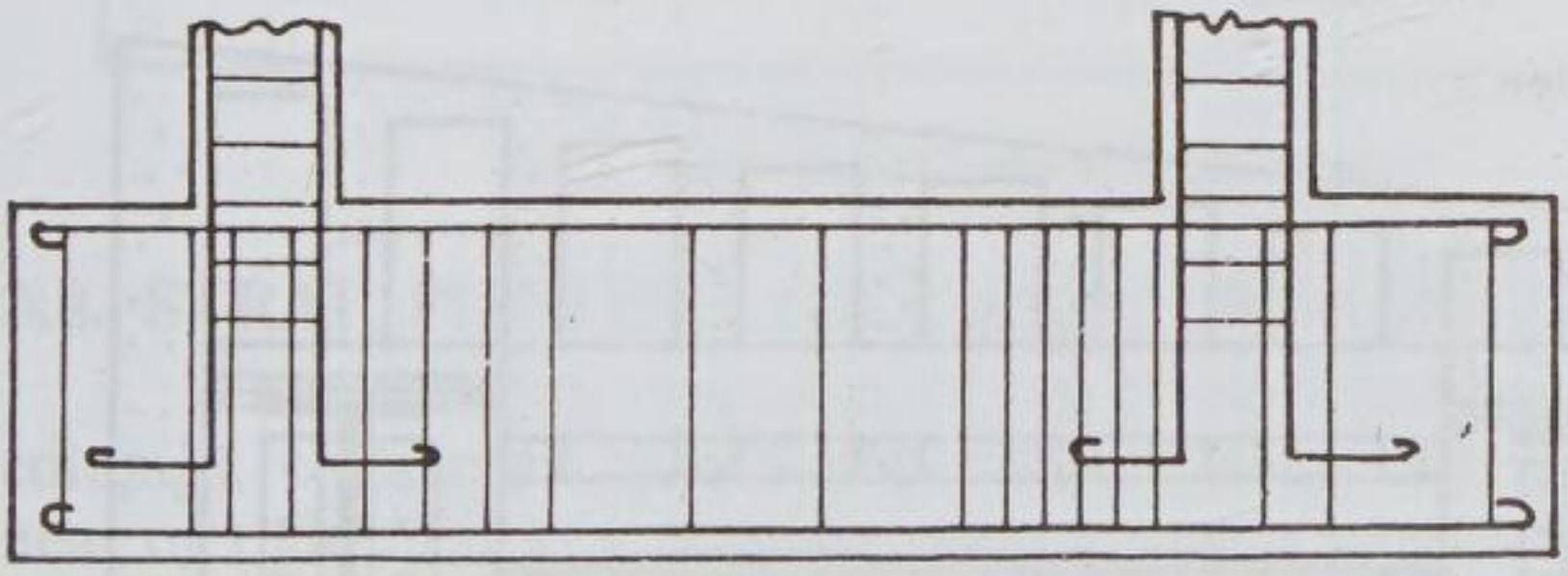


(a) L - Section

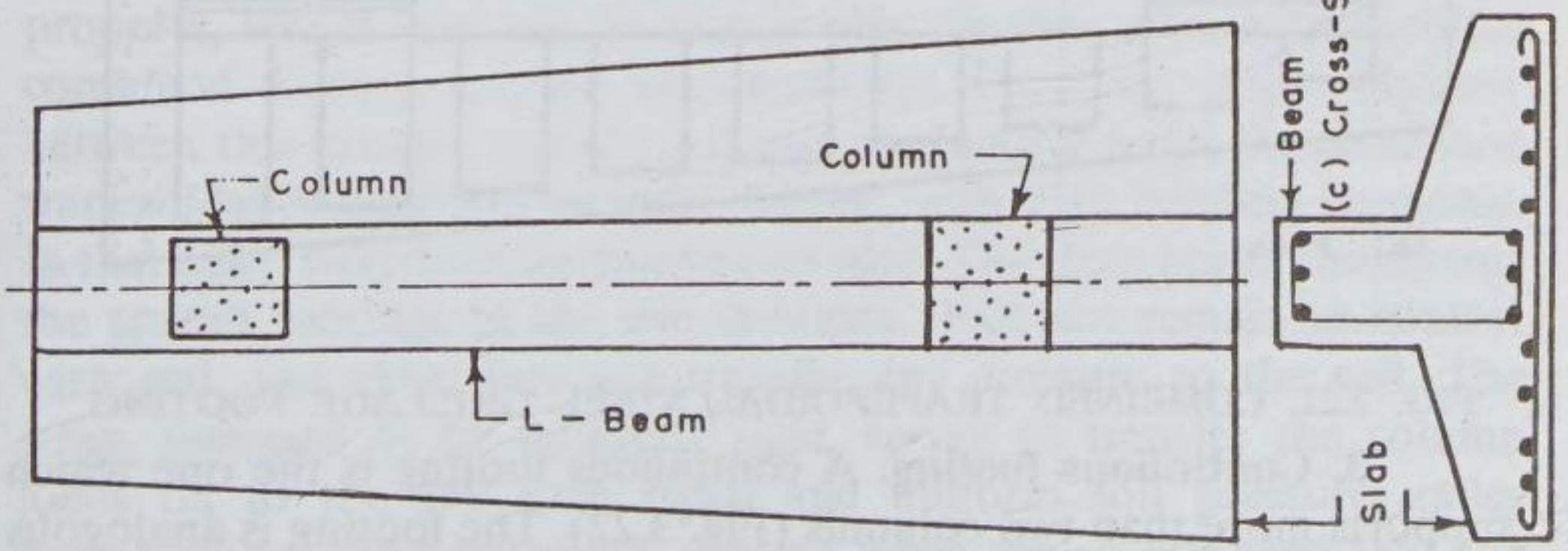


(b) Top Plan

FIG. 3.19. R.C.C. TRAPEZOIDAL FOOTING WITHOUT L-BEAM.



(a) L - Section Along Beam



(b) Top Plan

FIG. 3.20. R.C.C. TRAPEZOIDAL FOOTING WITH L-BEAM.

A combined trapezoidal footing may be either of R.C.C. or of steel grillage.

(*a*) ***Combined trapezoidal footing of R.C.C.*** The combined footing of R.C.C. may be either provided without longitudinal beam (Fig. 3.19) or it may have longitudinal beam (Fig. 3.20).

(*b*) ***Combined Trapezoidal grillage footing.*** Fig. 3.21 shows typical details of combined trapezoidal grillage footing. The top tier of joists is of uniform length, while the length and depth of joists in the bottom tier goes on increasing.

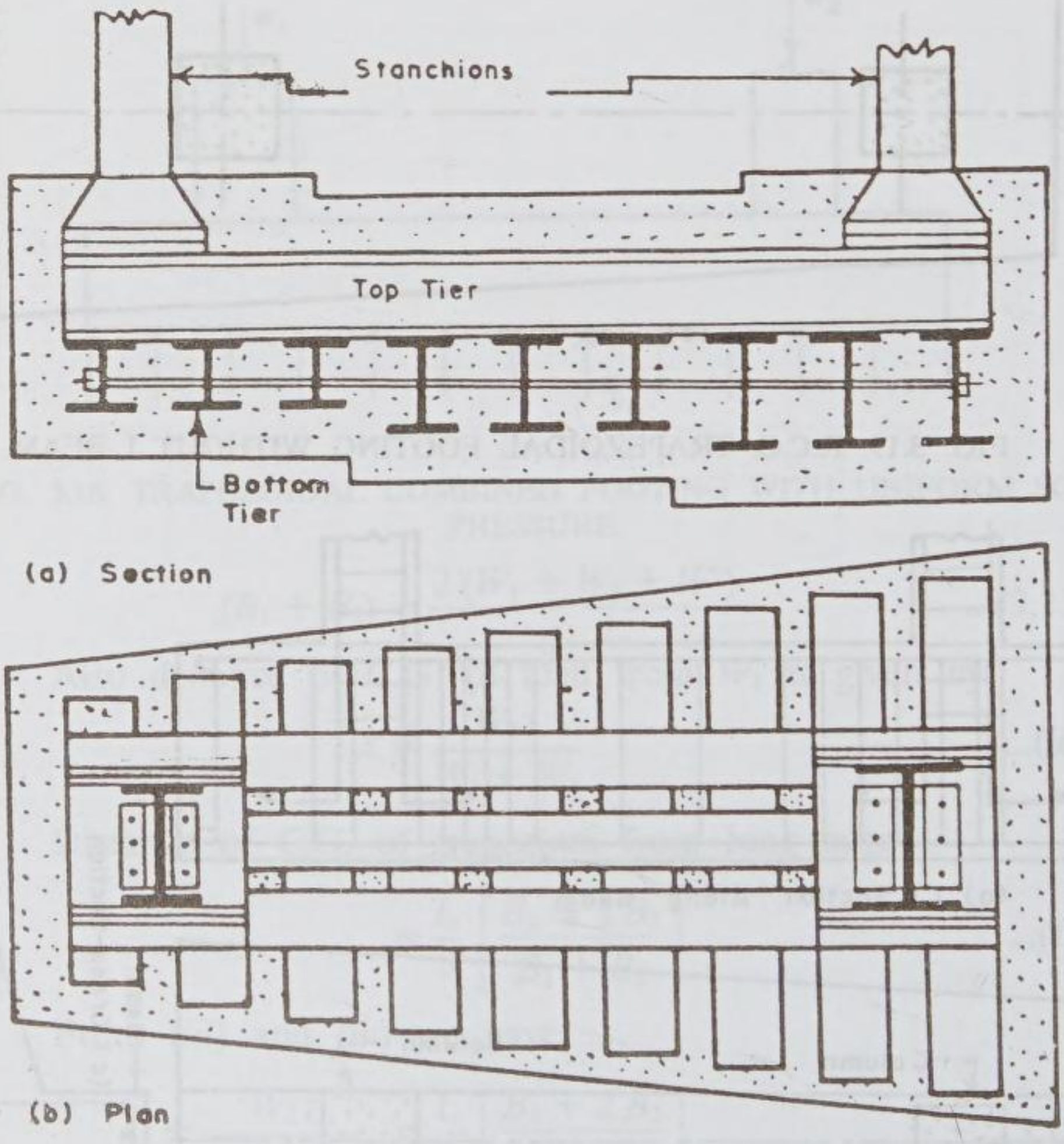


FIG. 3.21. COMBINED TRAPEZOIDAL STEEL GRILLAGE FOOTING.

3. Continuous footing. A continuous footing is the one which supports more than two columns (Fig. 3.22). The footing is analogous to the strip footing for wall. The loads from the individual columns are transferred either directly to the footing slab, or through a longitudinal beam running longitudinally when the loads are heavy.

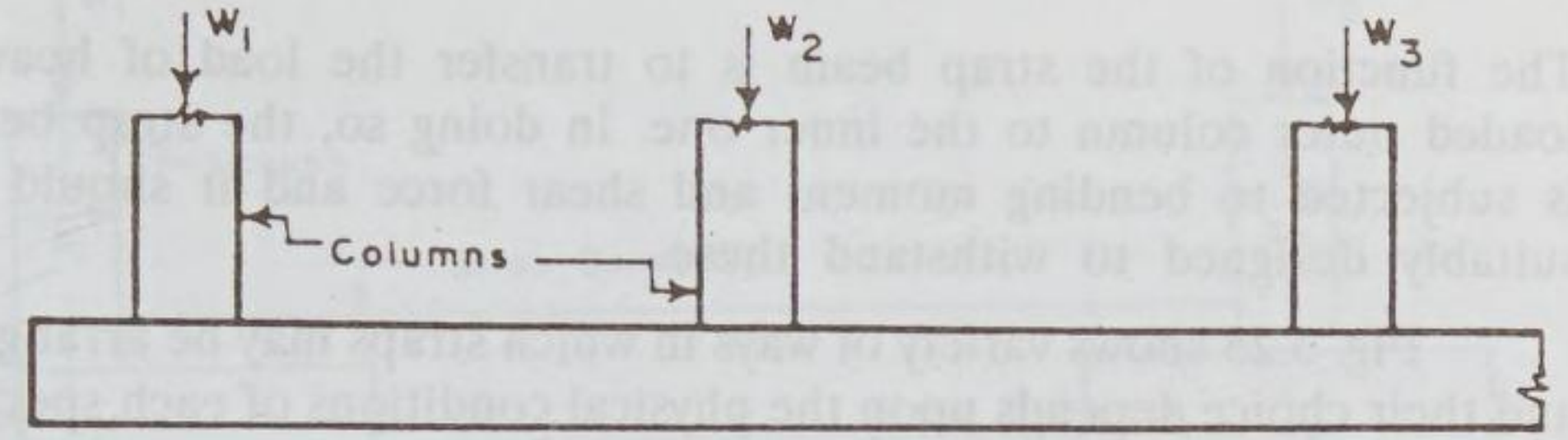


FIG. 3.22. CONTINUOUS FOOTING.

3.8. STRAP FOOTING OR CANTILEVER FOOTING

A strap footing comprises of two or more footings of individual columns, connected by a beam, called a strap. When a column is near or right next to a property limit, a square or rectangular footing concentrically located under the column would extend into the adjoining property, which may not be permissible. In that case, a trapezoidal combined footing may be an alternative. However, if the distance between this column and the adjoining column is *large*, the combined trapezoidal footing will be quite *narrow*, with high bending moments. In that case, strap footing may be provided. The strap beam, connecting the spread footings of the two columns, does not remain in contact with soil, and thus does not transfer any pressure to the soil. The strap, assumed to be infinitely rigid, serves to transfer the column loads on to the soil with equal and uniform soil pressure under both footings. The individual footing areas are so arranged that the C.G. of the combined loads of the two columns pass through the combined C.G. of the two footing areas. Once this criterion is achieved, the pressure distribution below each individual footing will be uniform.

The function of the strap beam is to transfer the load of heavily loaded outer column to the inner one. In doing so, the strap beam is subjected to bending moment and shear force and it should be suitably designed to withstand these.

Fig. 3.23 shows variety of ways in which straps may be arranged, and their choice depends upon the physical conditions of each specific case.

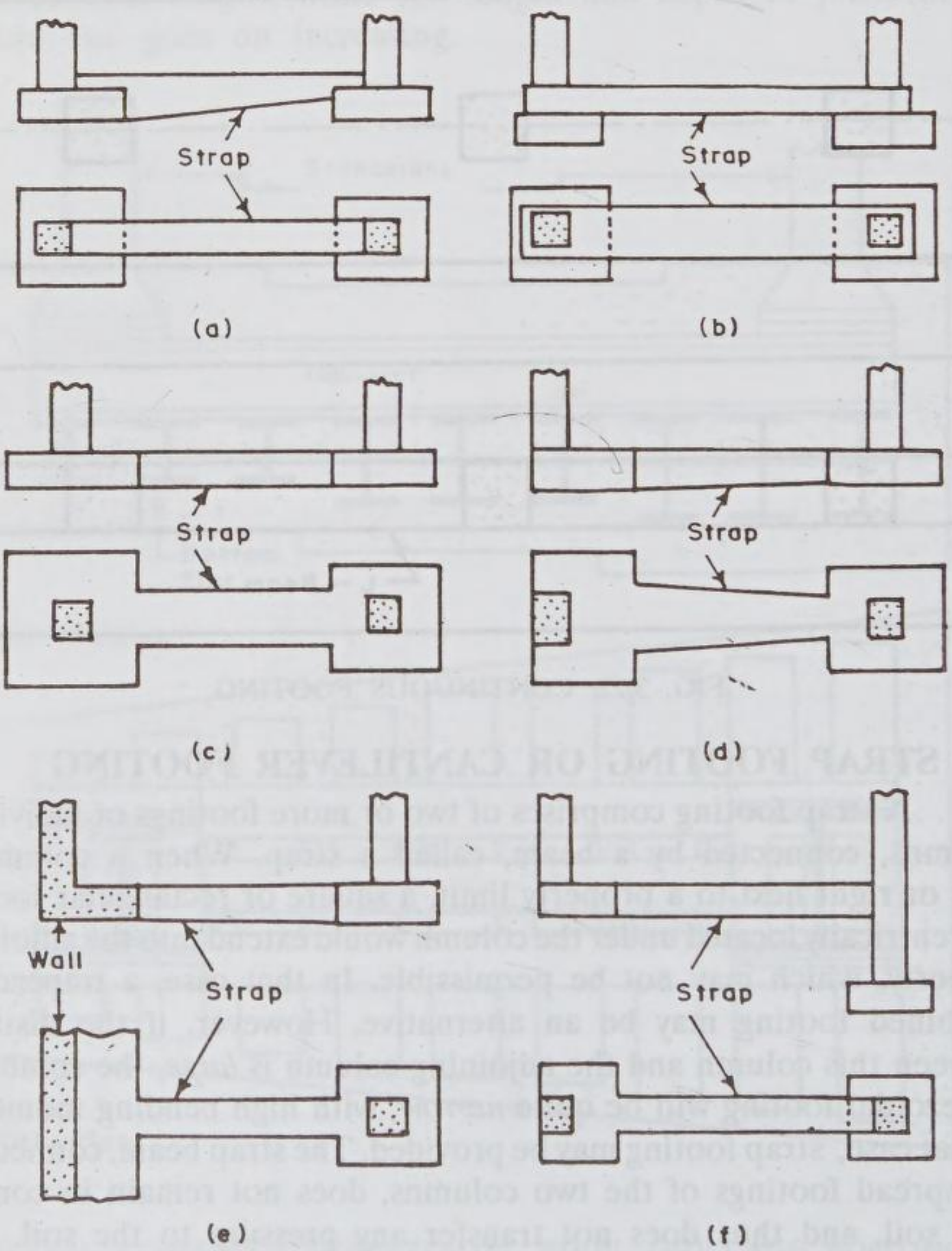


FIG. 3.23. COMMON ARRANGEMENTS OF STRAP FOOTINGS.

Proportioning of strap footing. Fig. 3.24 shows two columns A and B, transmitting axial loads W_1 and W_2 and are spaced l apart, centre to centre. Let W' be the total weight of both the individual footings. If A_1 and A_2 are the individual footing areas, and q_s is

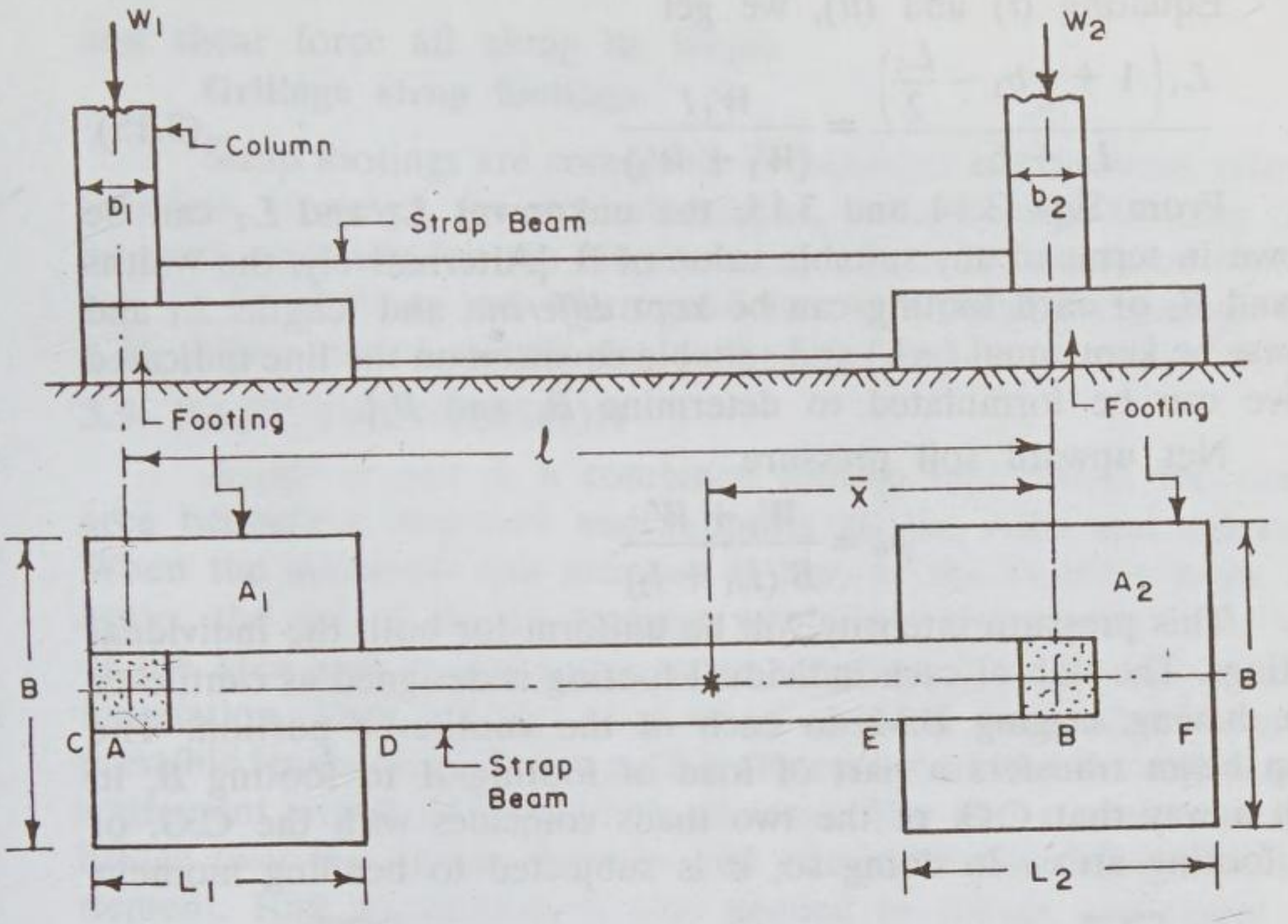


FIG. 3.24. STRAP FOOTING.

the safe bearing capacity of the soil, we have

$$A_1 + A_2 = \frac{W_1 + W_2 + W'}{q_s}$$

or
$$B\,(L_1 + L_2) = \frac{W_1 + W_2 + W'}{q_s} \qquad ...(3.14)$$

where B is the common width of each footing and L_1 and L_2 are the individual lengths of the footing. The length L_2 is arranged centrally under column B.

The C.G. of resultant load $W = W_1 + W_2$ falls at $\bar{x}$ from the centre of column B, given by

$$\bar{x} = \frac{W_1 l}{W_1 + W_2} \qquad ...(i)$$

Let $(b_1 \times b_1)$ and $(b_2 \times b_2)$ be the size of column A and B respectively. Taking moments of footing areas about the centre of column B, we get

$$\bar{x} = \frac{(B \times L_1)\left(l + \frac{1}{2}b_1 - \frac{L_1}{2}\right)}{B\,(L_1 + L_2)} \qquad ...(ii)$$

Equating (*i*) and (*ii*), we get

$$\frac{L_1\left(1+\frac{1}{2}b_1-\frac{L_1}{2}\right)}{L_1+L_2}=\frac{W_1 l}{(W_1+W_2)} \qquad ...(3.15)$$

From Eqs. 3.14 and 3.15, the unknowns L_1 and L_2 can be known in terms of any suitable value of B. [Alternatively, the widths B_1 and B_2 of each footing can be kept *different*, and lengths L_1 and L_2 may be kept equal ($=L$) and suitable equation on the line indicated above can be formulated to determine B_1 and B_2].

Net upward soil pressure

$$p_0=\frac{W_1+W_2}{B\,(L_1+l_2)}.$$

This pressure intensity will be uniform for both the individual footings. The slab of each individual footing is designed as cantilever slab, having sagging B.M. in each of the cantilever portion. The strap beam transfers a part of load of footing *A* to footing *B*, in such a way that C.G. of the two loads coincides with the C.G. of the footing areas. In doing so, it is subjected to bending moment

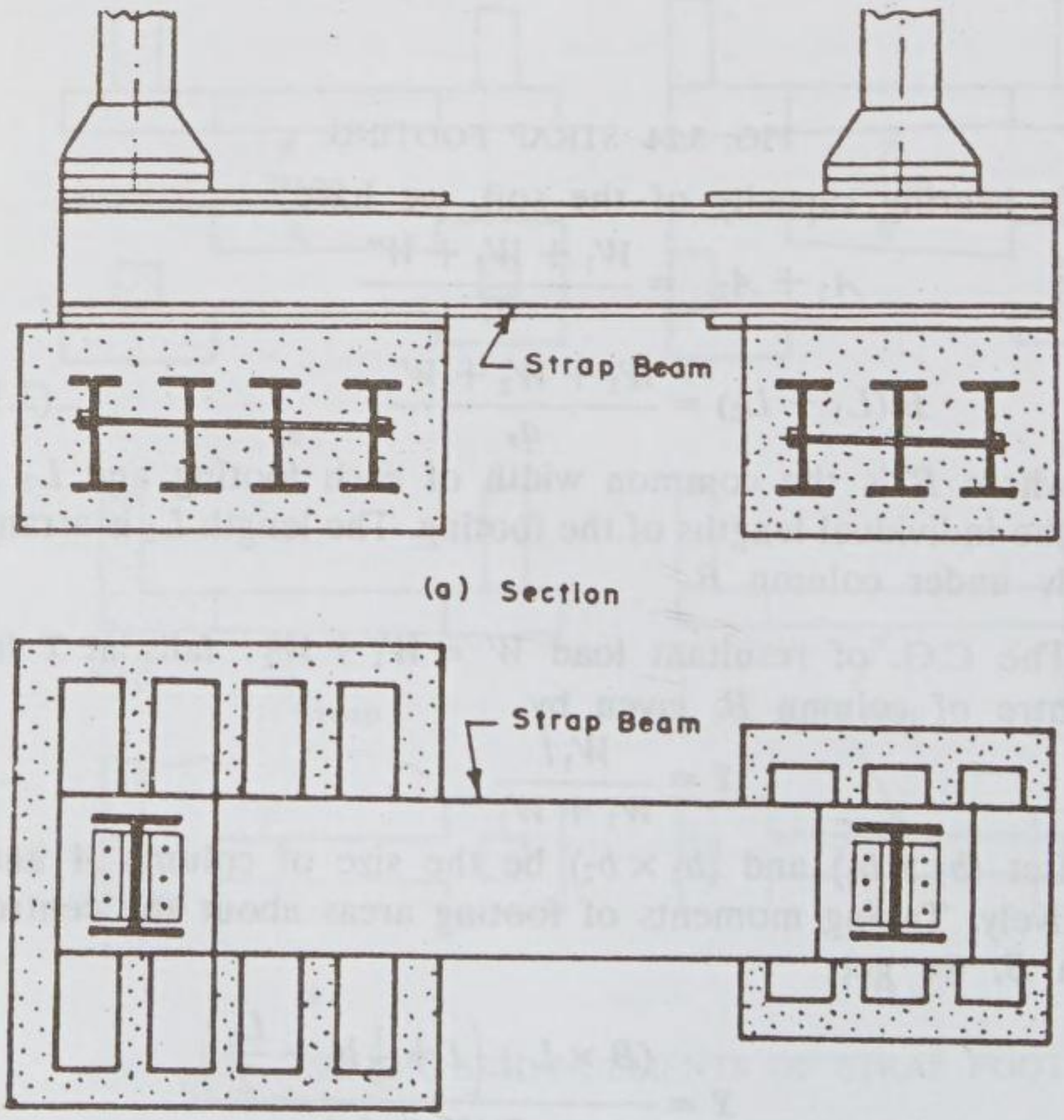


FIG. 3.25. GRILLAGE STRAP FOOTING.

and shear force all along its length.

Grillage strap footings.

Strap footings are commonly constructed in reinforced cement concrete. However, for steel stanchions, grillage strap footings may be used, specially for the circumstance where the depth of footing has to be shallow and where soil has low bearing capacity. Fig. 3.25 shows typical details for such a footing.

3.9. RAFT FOUNDATION

A *raft or mat* is a combined footing that covers the entire area beneath a structure and supports all the walls and columns. When the allowable soil pressure is low, or the building loads are heavy, the use of spread footings would cover more than one-half of the area and it may prove more economical to use mat or raft foundation. They are also used where the soil mass contains compressible lenses or the soil is sufficiently erratic so that the differential settlement would be difficult to control. The mat or raft tends to bridge over the erratic deposits and eliminates the differential settlement. Raft foundation is also needed to reduce settlement on highly compressible soils, by making the weight of structure and raft approximately equal to the weight of the soil excavated. Fig. 3.26 shows different types of raft or mat foundation.

A true raft or mat is a flat concrete slab with uniform thickness throughout the area, as shown in Fig. 3.26 (*a*). This is adopted only when the column spacing is small and column loads are also relatively small. If the column loads are heavy, the slab under the columns is thickened, as shown in Figs. 3.26 (*b*) and (*c*). If the column spacing is large, and/or the column loads are heavy, thickened bands may be provided along the column lines in both the directions. These bands are called main and secondary beams. If the loads are extremely heavy, two way grid structure made of cellular construction [Fig. 3.26 (*e*)] may be used. Where basements are to be provided, the basement walls may be used as ribs or deep beams [Fig. 3.25 (*f*)]. A raft often rests directly on soil or rock. However, it may also rest on piles.

Ordinarily, rafts are designed as reinforced concrete flat slabs. If the C.G. of loads coincide with the centroid of the raft, the upward load is regarded as a uniform pressure equal to the downward load divided by the area of the raft. The weight of the raft is not considered in the structural design because it is assumed to be carried directly by the subsoil. Since this method does not take into account moments and shears caused by differential settlements, it is customary to reinforce the raft more heavily than required according to the analysis.

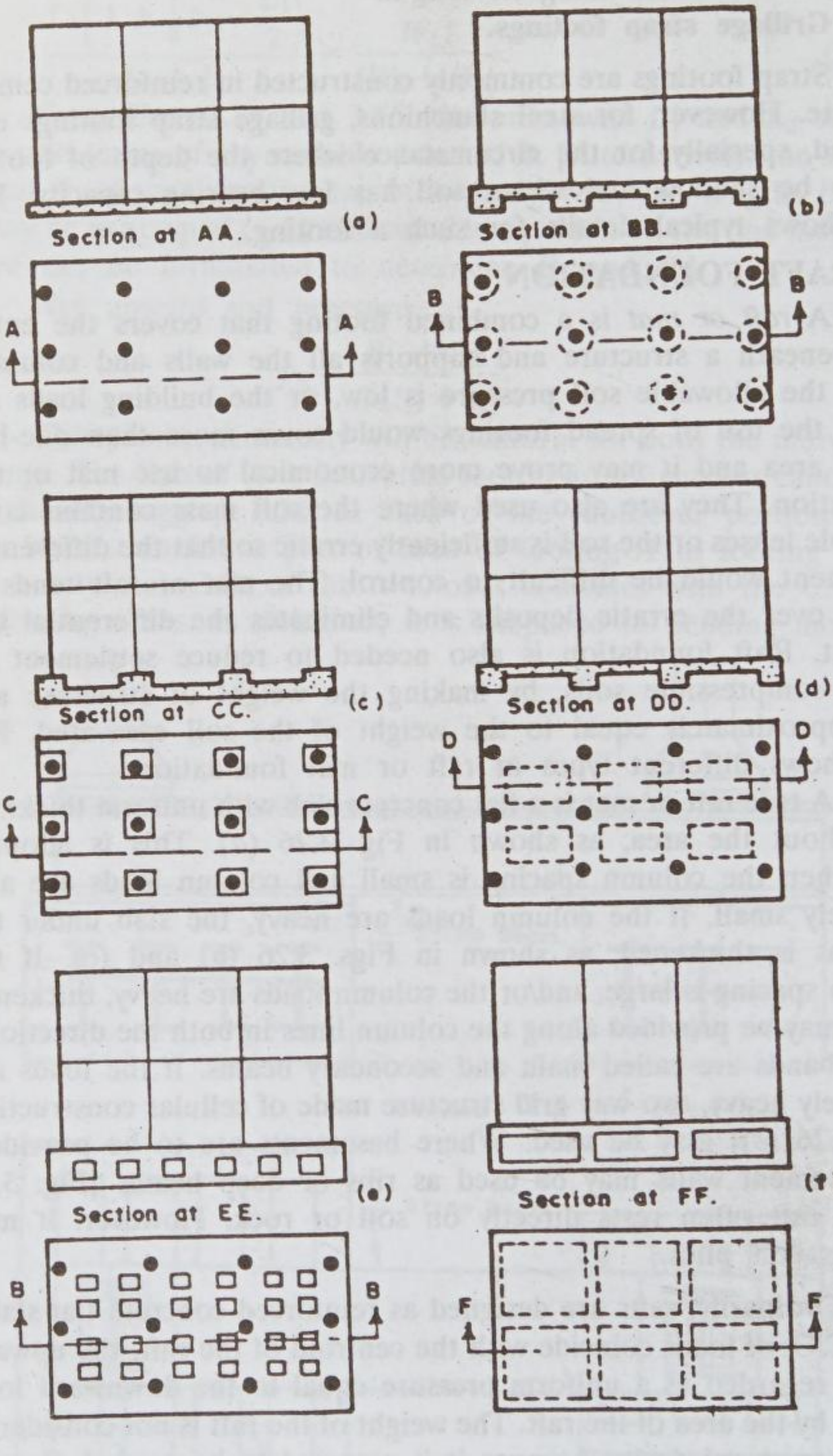


FIG. 3.26. COMMON TYPES OF RAFT FOUNDATIONS.

(*a*) Flat plate type (*b*) Flat plate thickened under columns (*c*) Flat plate with padestals (*d*) Two way beam and slab type (*e*) Cellular construction (*f*) Basement walls as rigid frame.

A raft may undergo large settlements without causing harmful differential settlement. For this reason, almost double the settlement of that permitted for footings is acceptable for raft. If a maximum settlement of 5 cm is permitted for a raft, the differential settlement is not likely to exceed 2 cm. Based on this consideration, the net pressure can be calculated from the following equation for rafts having width greater than 6 m:

$$q_p = 2\,(N - 3)\,R_{w2}\ (\text{t/m}^2) \qquad ...[3.16\ (a)]$$

or $$q_p = 20\,(N - 3)\,R_{w2}\,(\text{kN/m}^2) \qquad ...[3.16(b)]$$

The penetration resistance N value should be taken at 75 cm intervals for depths equal to width of the raft, below the base of the raft. The minimum average value of N for the holes should be used in the above equations. If N is less than 5, sand should be compacted by artificial means to raise N above 10, or else piles or piers should be used.

Example 3.3. *Find the dimensions of a combined rectangular footing for two columns A and B, carrying loads of 50 t and 70 t respectively. Column A is 30 cm × 30 cm in size and column B is 40 cm × 40 cm in size. The centre to centre spacing of the columns is 3.4 metres. The safe bearing capacity of the soil may be taken as 15 tonnes/m².*

Solution. Refer Fig. 3.14.

Given $W_1 = 50$ t ; $W_2 = 70$ t ;

$$l = 3.4\ \text{m}\ ;\ q_s = 15\ \text{t/m}^2$$

Let the weight of footing $= W' = 10\ \%$ of $(W_1 + W_2) = 12$ t.

$$\therefore \qquad A = \frac{W_1 + W_2 + W'}{q_s} \quad \text{(Eq. 3.8)}$$

$$= \frac{50 + 70 + 12}{15} = 8.8\ \text{m}^2$$

Let the size of the footing be 1.8 m × 5 m. (*i.e.* $B = 1.8$ m and $L = 5$ m).

The projections a_1 and a_2 should be such that C.G. of footing coincide with the C.G. of column loads. The distance $\bar{x}$ of the C.G. of column loads from the centre of column A is given by

$$\bar{x} = \frac{W_2 l}{W_1 + W_2} = \frac{70 \times 3.4}{50 + 70} = 2\ \text{m.}$$

$$\therefore \qquad a_1 + \bar{x} = \frac{L}{2}$$

or $$a_1 = \frac{L}{2} - \bar{x} = \frac{5}{2} - 2 = 0.5\ \text{m.}$$

Also. $$a_2 = L - (l + a_1) = 5 - (3.4 + 0.5) = 1.1\ \text{m.}$$

The dimensions of the footing are shown in Fig. 3.27. The net upward pressure p_0 is given by

$$p_0 = \frac{W_1 + W_2}{b \times L} = \frac{50 + 70}{1.8 \times 5} = 13.33 \text{ t/m}^2$$

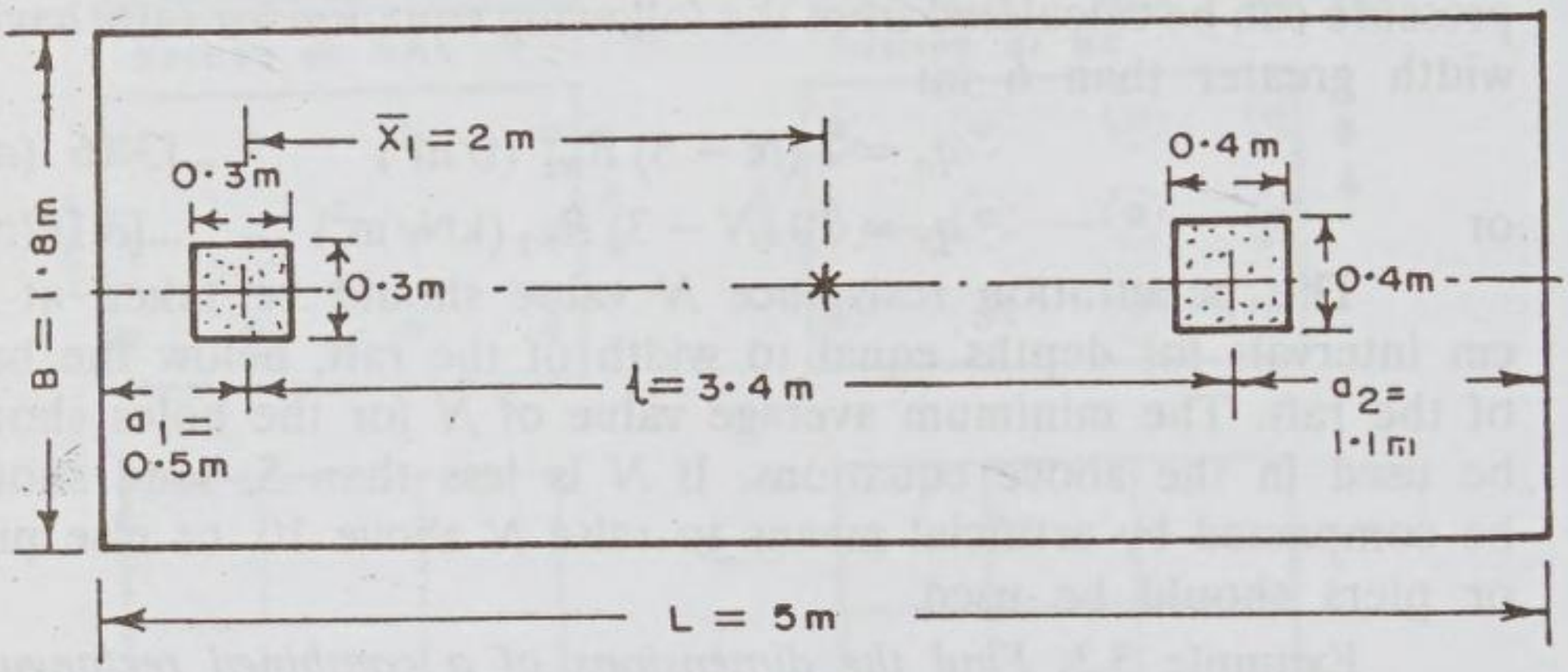


FIG. 3.27.

This uniform pressure intensity is used for the structural design of the footing slab.

Example 3.4. *Find the dimensions of a combined trapezoidal footing for two columns A and B, spaced 5 metres centre to centre. Column A is 40 cm × 40 cm in size and transmits a load of 90 tonnes. Column B is 30 cm × 30 cm in size and carries a load of 60 tonnes. The maximum length of footing is restricted to 7 metres only. The safe bearing capacity of soil may be taken as 12 tonnes/m².*

Solution. Refer Fig. 3.18.

Given $W_1 = 90$ tonnes; $W_2 = 60$ tonnes;

$l = 5$ m ; $q_s = 12$ tonnes/m²

Keep total length L equal to the maximum available length

$= 7$ m.

Keep equal projections, such that

$$a_1 = a_2 = \frac{1}{2}(7 - 5) = 1 \text{ m.}$$

Let the widths of the footing be B_1 and B_2 . The values of B_1 and B_2 should be such that C.G. of footing coincides with C.G. of the column loads.

Let the weight of footing be

$$W' = 0.1 (W_1 + W_2) = 0.1 (90 + 60) = 15 \text{ t.}$$

$$\therefore \quad \frac{(B_1 + B_2)L}{2} = \frac{W_1 + W_2 + W'}{q_s}$$

or $$(B_1 + B_2) = \left(\frac{90 + 60 + 15}{2}\right) \times \frac{2}{7} = 3.93 \text{ m} \quad ...(1)$$

Distance $\bar{x}$ of the C.G. of the loads, from W_1 is

$$\bar{x} = \frac{W_2 l_1}{W_1 + W_2} = \frac{60 \times 5}{90 + 60} = 2 \text{ m.}$$

Distance of C.G. of loads from edge of B_1

$$= a_1 + \bar{x} = 1 + 2 = 3 \text{ m} \qquad ...(i)$$

Distance of C.G. of footing from edge B_1

$$= \frac{L}{3}\left(\frac{B_1 + 2B_2}{B_1 + B_2}\right)$$

$$= \frac{7}{3}\left(\frac{B_1 + 2B_2}{B_1 + B_2}\right) \qquad ...(ii)$$

Equating (*i*) and (*ii*), we get

$$\frac{7}{3}\left(\frac{B_1 + 2B_2}{B_1 + B_2}\right) = 3$$

or

$$\frac{B_1 + 2B_2}{B_1 + B_2} = \frac{9}{7} \qquad ...(2)$$

Solving (1) and (2), we get

$$B_1 = 2.81 \text{ m and } B_2 = 1.12 \text{ m}$$

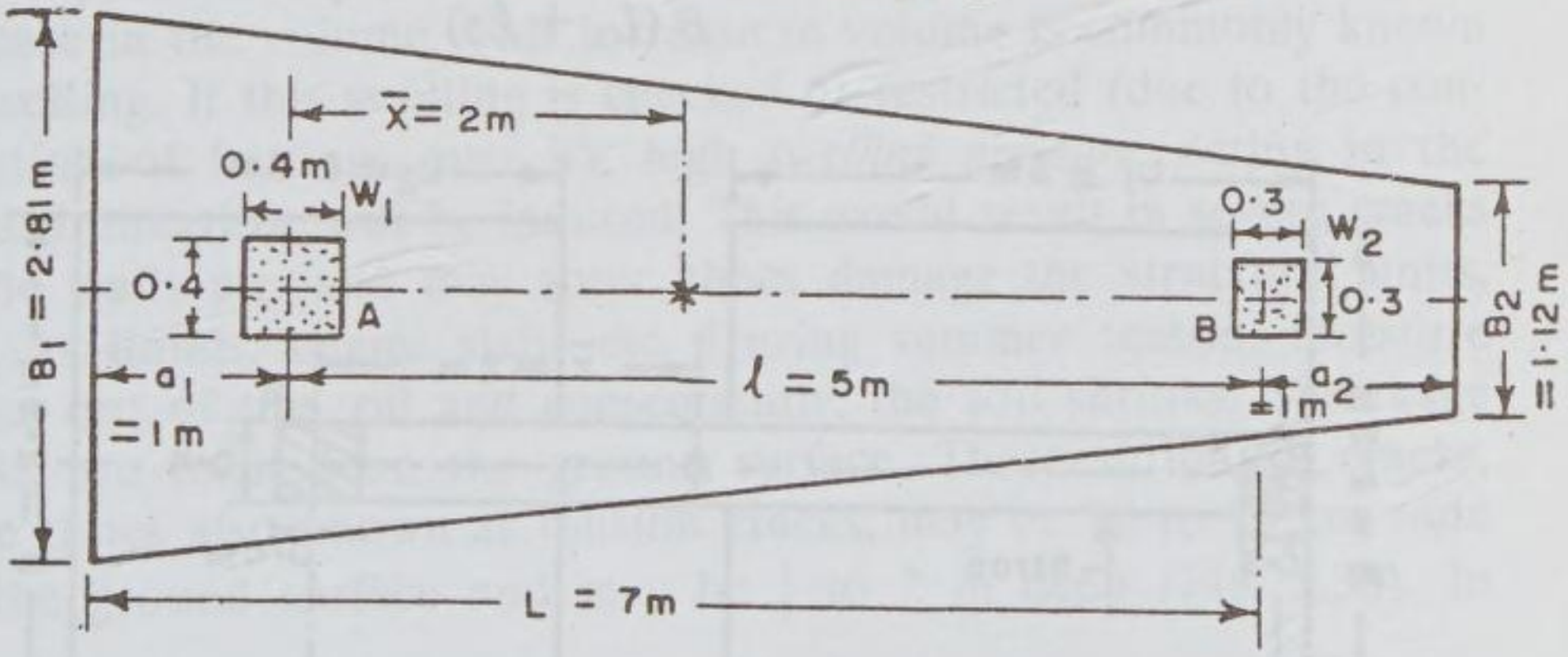


FIG. 3.28.

The footing, fully dimensioned, is shown in Fig. 3.28. The net upward soil pressure intensity is given by

$$p_0 = \frac{W_1 + W_2}{\frac{1}{2}(B_1 + B_2)\,L}$$

$$= \frac{90 + 60}{\frac{1}{2}(2.83 + 1.12)\,7} \approx 11 \text{ t/m}^2$$

Example 3.5. *Find the dimensions of a strap footing for two columns A and B, spaced 5 metres centre to centre. Column A, 30 cm × 30 cm carries a load of 600 kN and is on the property line. Column B, 40 cm × 40 cm in size carries a load of 900 kN. The bearing capacity of soil is 120 kN/m².*

Solution. Refer Fig. 3.24 for the general arrangement of the footing. Let the width of two spread footing be B metres each. Let the length of footing under column A be L_1 and that under column B be L_2 centrally arranged under B.

Given $W_1 = 600$ kN; $W_2 = 900$ kN.

Let weight of footing, $W' = 10\,\%$ of(W_1+W_2) $=0.1(600+900)$

$= 150$ kN.

Hence $B\,(L_1 + L_2) = \dfrac{600 + 900 + 150}{120} = 13.75\ \text{m}^2$

or $$L_1 + L_2 = \frac{13.75}{B} \qquad ...(1)$$

Let $\bar{x}$ = distance of C.G. of loads from centre of column B.

$$\bar{x} = \frac{W_1 l}{W_1 + W_2} = \frac{600 \times 5}{600 + 900} = 2 \text{ m.}$$

If $\bar{x}$ is also the distance of C.G. of areas, from the centre of column B, we have

$$\bar{x} = \frac{B \times L_1 \left(l + \dfrac{b_1}{2} - \dfrac{L_2}{2}\right)}{B\,(L_1 + L_2)}$$

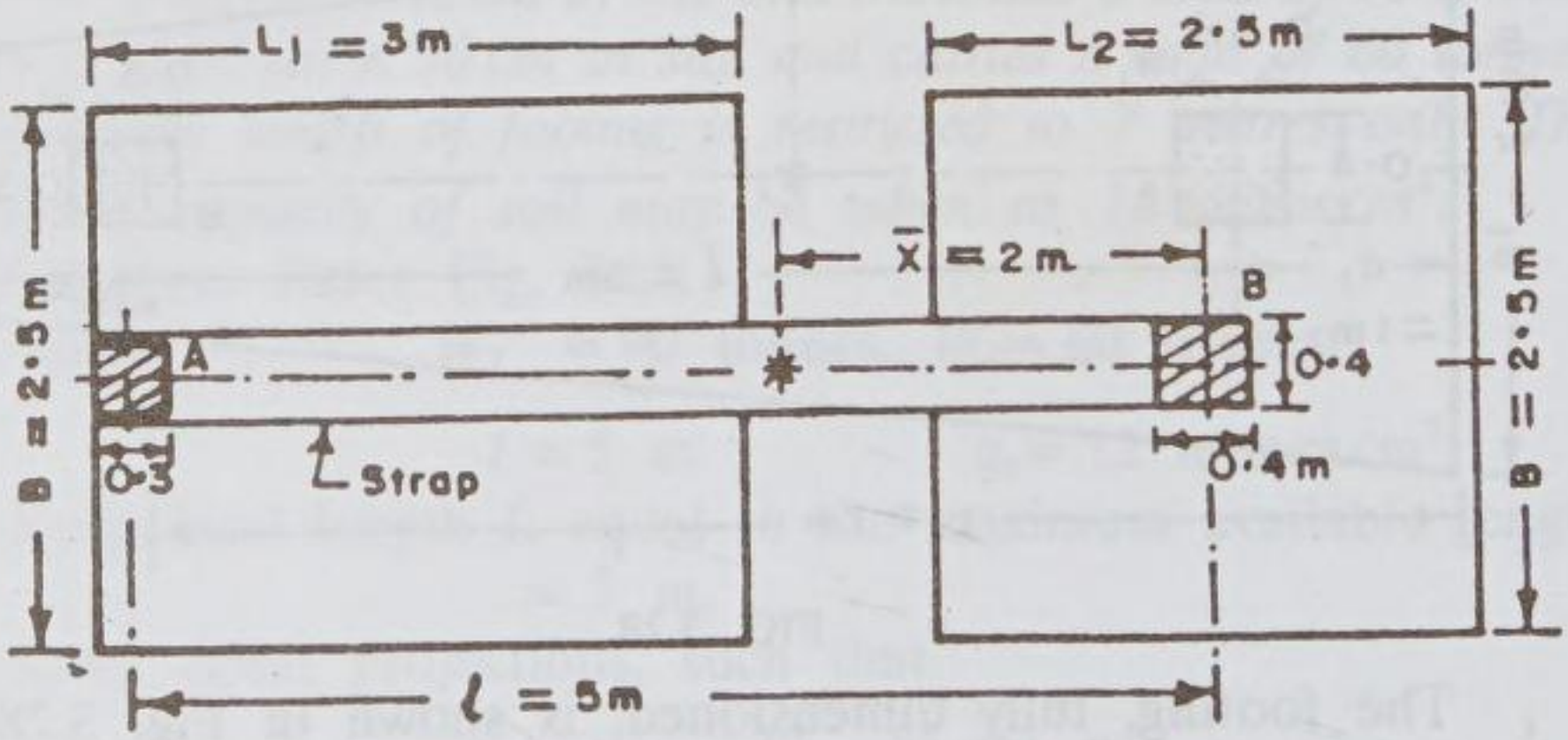


FIG. 3.29.

Substituting the values of $\bar{x}$, b_1 and l_1, we get

$$2 = \frac{L_1 (5 + 0.15 - 0.5\,L)}{(L_1 + L_2)} \qquad ...(2)$$

Substituting the value of $(L_1 + L_2)$ from (1), and choosing $B = 2.5$ m, we get

$$\frac{L_1 (5.15 - 0.5\,L_1)}{13.75/2.5} = 2$$

$$L_1^2 - 10.3\,L_1 + 22 = 0$$

From which $L_1 = 3.052 \approx 3$ m (say)

$$\therefore \qquad L_2 = \frac{13.75}{2.5} - 3.0 = 2.5 \text{ m.}$$

The general arrangement of footings is shown in Fig. 3.29.

Net upward soil pressure

$$p_0 = \frac{W_1 + W_2}{B\,(L_1 + L_2)} = \frac{600 + 900}{2.5\,(3 + 2.5)}$$

$$= \mathbf{109\ kN/m^2}$$

The structural design of individual footing slab will be done for the above *uniform* soil pressure.

3.10. FOUNDATIONS FOR BLACK COTTON SOILS

Black cotton soils and other *expansive* soils have typical characteristics of shrinkage and swelling due to moisture movement through them. During rainy season, moisture penetrates into these soils, due to which they swell. Most of the fine grained clays, including black cotton soils have their grains which are more or less in the form of platelets or sheets (just like leafs of a book), and their grains are not round. When moisture enter between the platelets under some hydrostatic pressure, the particles separate out, resulting in increase in the volume. This increase in volume is commonly known as swelling. If this swelling is checked or restricted (due to the construction of footings over it), high *swelling pressure,* acting in the upward direction, will be induced. This would result in severe cracks in the walls etc. and may some times damage the structural units, such as lintels, beams slabs etc. During summer season, moisture moves out of the soil and consequently, the soil shrinks. Shrinkage cracks are formed on the ground surface. These shrinkage cracks, some times also known as tension cracks, may be 10 to 15 cm wide on the ground surface and may be $\frac{1}{2}$ to 2 m deep (Fig. 3.30). In

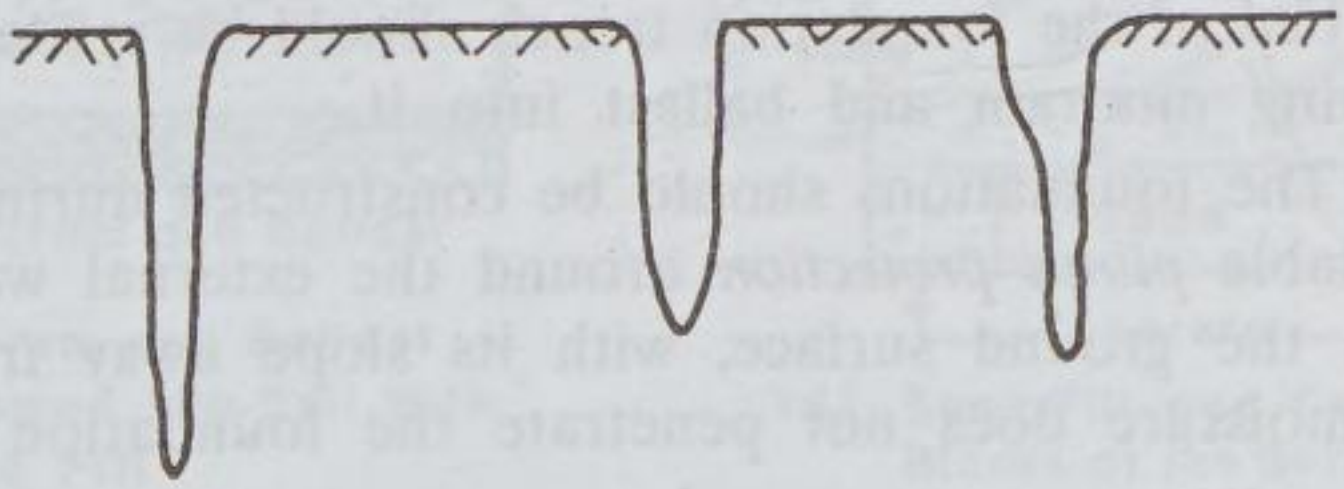


FIG. 3.30. FORMATION OF TENSION CRACKS IN EXPANSIVE SOIL.

fat clays, having angle of internal friction $\phi = 0$, the depth z of tension cracks is found to be equal $2c/\gamma$, where c is the unit cohesion and γ is the unit weight of the soil. These cracks result in loss of support beneath the footings, resulting in high settlements. Some expansive and shrinkable soils stick to the footing base and

pull the footing down when they shrink. This results in horizontal cracks in the walls and other flexible units of the structrure.

Black cotton soils and other expansive soils are dangerous due to their shrinkage and swelling characteristics. In addition to this, these soils have very poor bearing capacity, ranging from 5 t/m^2 to 10 t/m^2. In designing footings on these soils, the following points should be kept in mind:

1. The safe bearing capacity should be properly determined, taking into account the effect of sustained loading. The long term effect of loading results in slow consolidation. In absence of tests, the bearing capacity of these soils may be limited to 5 to 10 t/m^2.

2. The foundation should be taken at least 50 cm lower than the depth of moisture movement. This depth should also be much more than depth of tension cracks.

3. Where this soil occurs only in top layer, and where the thickness of this layer does not exceed 1 to 1.5 m, the entire layer of black cotton soil (or other expansive soil) should be removed, and the foundation should be laid on non-shrinkable non- expansive soil.

4. Where the depth of clay layer is large, the foundation or footing should be prevented from coming in contact with the soil. This can be done by excavating wider and deeper foundation trench and interposing layer of sand/mooram around and beneath the footing.

5. Where the soil is highly expansive, it is very essential to have minimum contact between the soil and the footing. This can be best achieved by transmitting the loads through deep piles or piers and by supporting wall loads on capping beams which are kept some distance (5 to 15 cm) above the ground surface, to permit free expansion of the soil

6. Where the bearing capacity of soil is poor, or soil is very soft, the bed of the foundation trench should be made firm or hard by ramming mooram and ballast into it.

7. The foundations should be constructed during dry season. Also suitable *plinth protection* around the external wall should be made on the ground surface, with its slope away from the wall, so that moisture does not penetrate the foundation during rainy season.

Types of foundation in black cotton soils.

Foundation in black cotton soils may be of the following types:

1. Strip or pad foundation.
2. Pier foundation
3. Under-reamed pile foundation.

1. Strip or pad foundation. For medium loads, strip foundation (for walls) and pad foundation (for columns) may be provided, along with special design features discussed above. Fig. 3.31 shows some typical sections of shallow footings suitable for black cotton and other expansive soils.

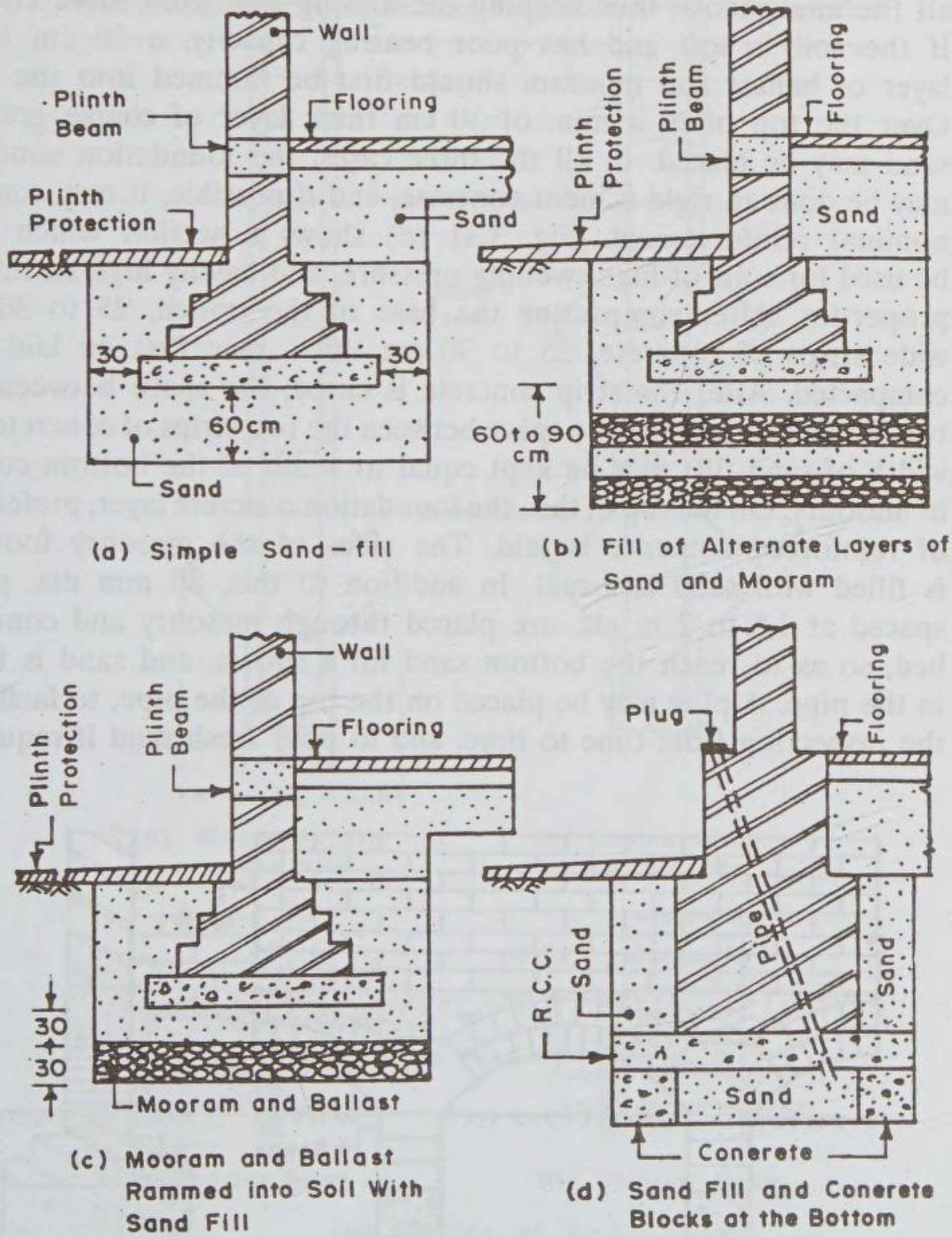


FIG. 3.31. STRIP FOOTING WITH SPECIAL TREATMENT.

Section of Fig. 3.31 (*a*) is suitable when the soil, though expansive, has little swelling pressure. A 60 cm thick layer of cohesionless sand is placed below the foundation concrete, and is compacted. Sand is also filled around the footing. When the soil swells, the sand grains would yield by moving up, thus relieving the swelling

pressure. When the soil shrinks, the sand layer would expand, but there will be no discontinuity in the soil support. Sand fill should also be used below flooring. Section of Fig. 3.31 (*b*) is suitable where the swelling pressures are relatively high. The alternate layers of mooram (or ballast) and sand act as a spring which can compress or expand along with the sub-soil movements. It will, thus absorb all the movements, thus keeping the footing free from these effects. If the soil is soft and has poor bearing capacity, a 30 cm thick layer of ballast and mooram should first be rammed into the soil. Over the top of it, a min. of 30 cm thick layer of coarse grained sand may be placed. In all the three cases, the foundation concrete may be done in rigid cement concrete, and if possible, it may contain nominal reinforcement. Fig. 3.31 (*d*) shows a section which may be used for soils of high swelling pressure, and having high shrinkage properties. After compacting the base of the tranch, 25 to 30 cm wide strips of concrete, 25 to 30 cm thick, may first be laid and compacted. After the strip concrete is cured, the space between the two is filled with sand. The space between the two strips of concrete(i.e. width of sand fill) may be kept equal to width of the bottom course of masonry. On the top of this, the foundation concrete layer, preferably of reinforced concrete is laid. The sides of the masonry footings is filled with sand as usual. In addition to this, 80 mm dia. pipes spaced at 1.5 to 2 m etc. are placed through masonry and concrete bed, so as to reach the bottom sand fill a shown, and sand is filled in the pipe. A plug may be placed on the top of the pipe, to facilitate the inspection from time to time, and to pour fresh sand if required.

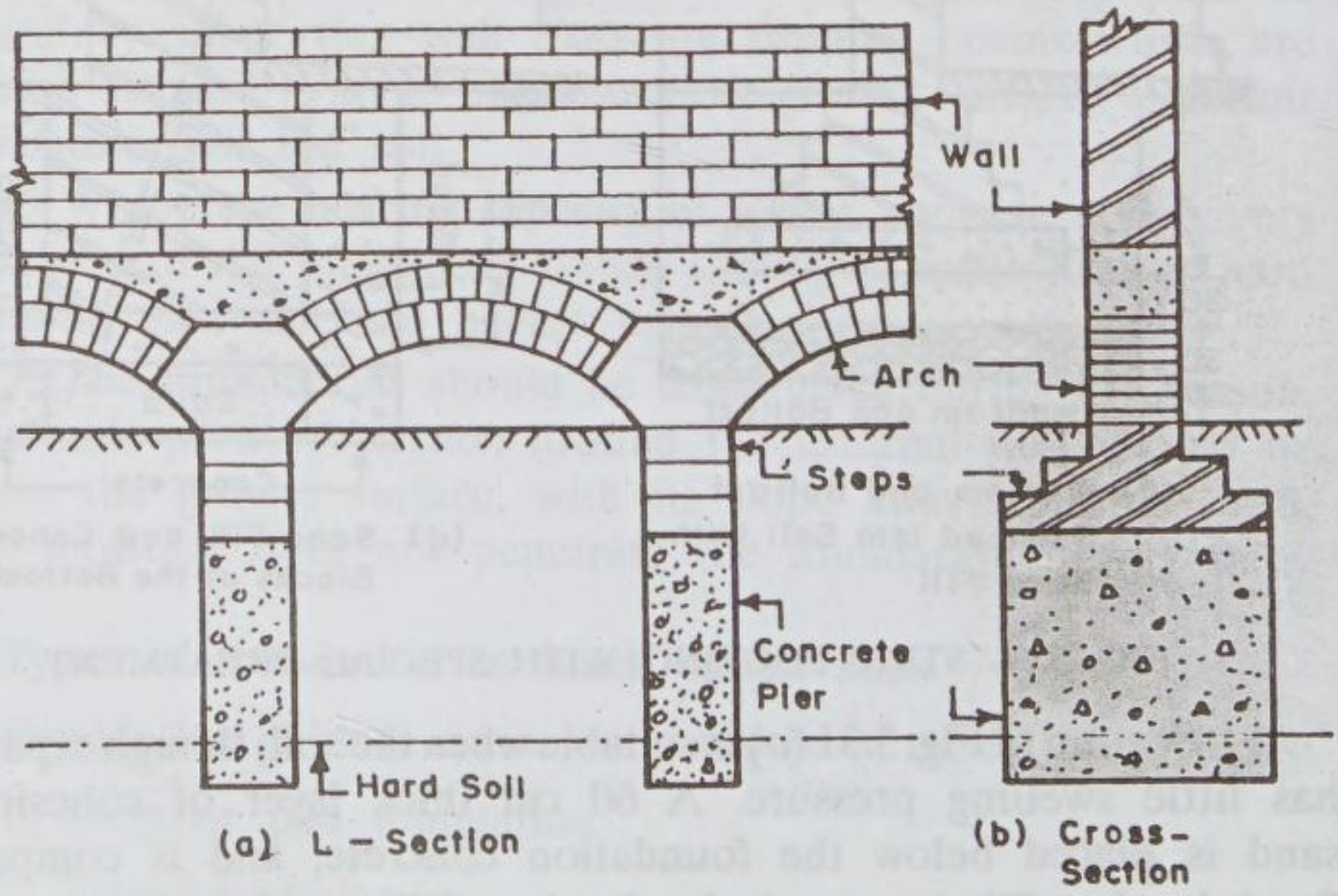


FIG. 3.32. PIER FOUNDATIONS WITH ARCH.

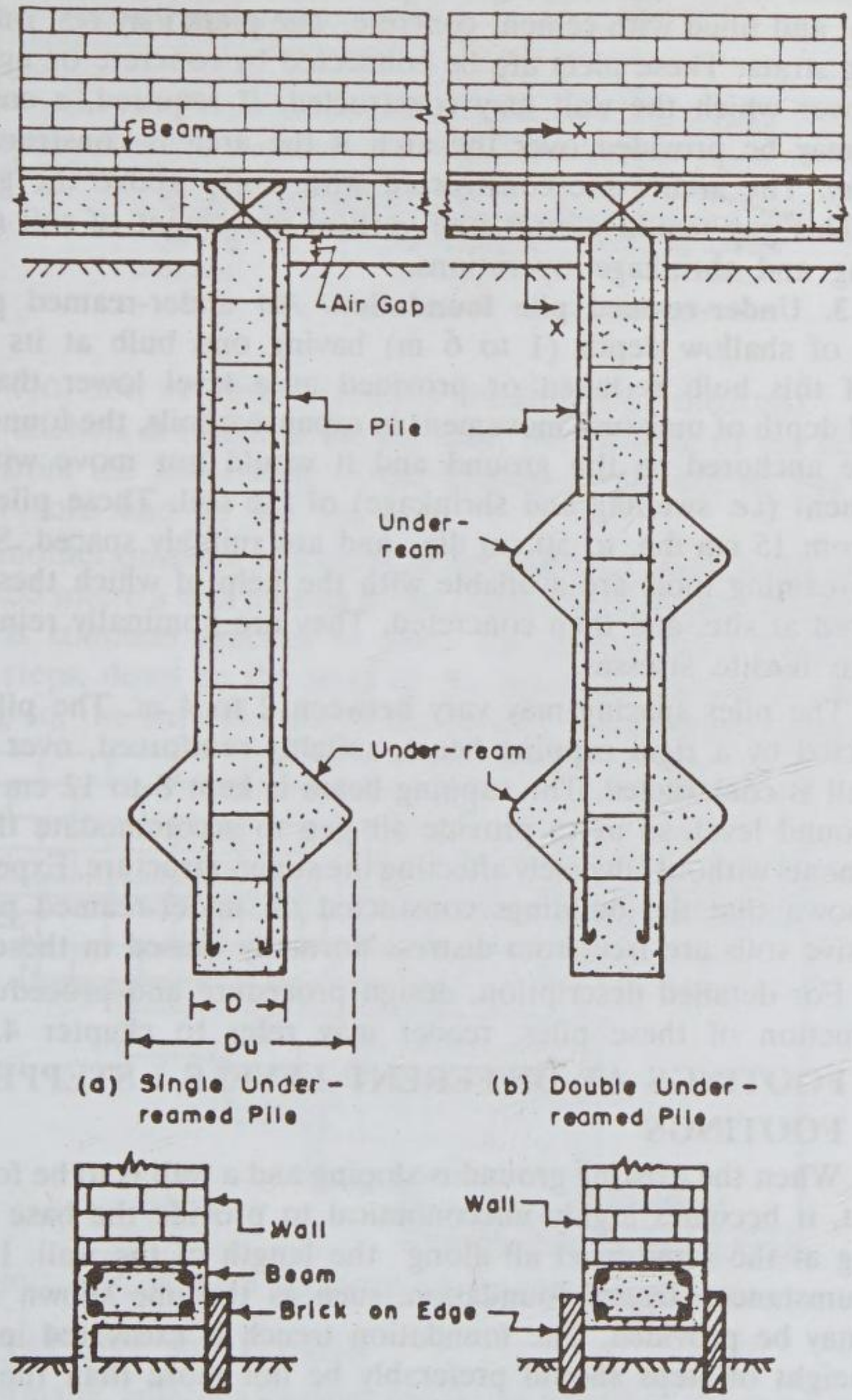


FIG. 3.33. UNDER-REAMED PILE FOUNDATION.

2. Pier foundation with arches. Fig. 3.32 shows a typical pier foundation for a wall carrying heavy loads. Piers are dug at regular interval and filled with cement concrete. The piers may rest on good bearing strata. These piers are be connected by concrete or masonry arch, over which the wall may constructed. If required, a concrete beam may be provided over the arch if the arch is constructed of masonry. The arches are constructed with a gap above the ground level. This gap would permit free vertical movement of soil during swelling and shrinkage operations.

3. Under-reamed pile foundation. An under-reamed pile is a pile of shallow depth (1 to 6 m) having one bulb at its lower end. If this bulb is taken or provided at a level lower than the critical depth of moisture movement in expansive soils, the foundation will be anchored to the ground and it would not move with the movement (*i.e.* swelling and shrinkage) of the soil. These piles may vary from 15 cm dia. to 50 cm dia., and are suitably spaced. Special under-reaming tools are available with the help of which these may be bored at site, and then concreted. They are nominally reinforced to take tensile stresses.

The piles spacing may vary between 2 to 4 m. The piles are connected by a rigid capping beam, suitably reinforced, over which the wall is constructed. The capping beam is kept 8 to 12 cm above the ground level, so as to provide air gap to accommodate the soil movements without adversely affecting the super- structure. Experience has shown that the buildings constucted on under-reamed piles in expansive soils are free from distress normally caused in these soils.

For detailed description, design procedure and procedure for construction of these piles, reader may refer to chapter 4.

3.11. FOOTINGS AT DIFFERENT LEVELS : STEPPED FOOTINGS

When the existing ground is sloping and a wall is to be founded over it, it becomes highly uneconomical to provide the base of the footing at the same level all along the length of the wall. In such a circumstance, *stepped* foundation, such as the one shown in Fig. 3.34 may be provided. The foundation trench is excavated in steps. The height of steps should preferably be not more than the depth of the concrete block and each step should be a multiple of the thickness of brick or stone course. The overlap between two layers of foundation concrete should be less than the vertical thickness of concrete.

According to the National Building Code, the distance of the sloping surface at the base level of footing to the centre of the footing should not be nearer then twice the width of the footing for normal loadindgs. When footings are heavily loaded, a slope

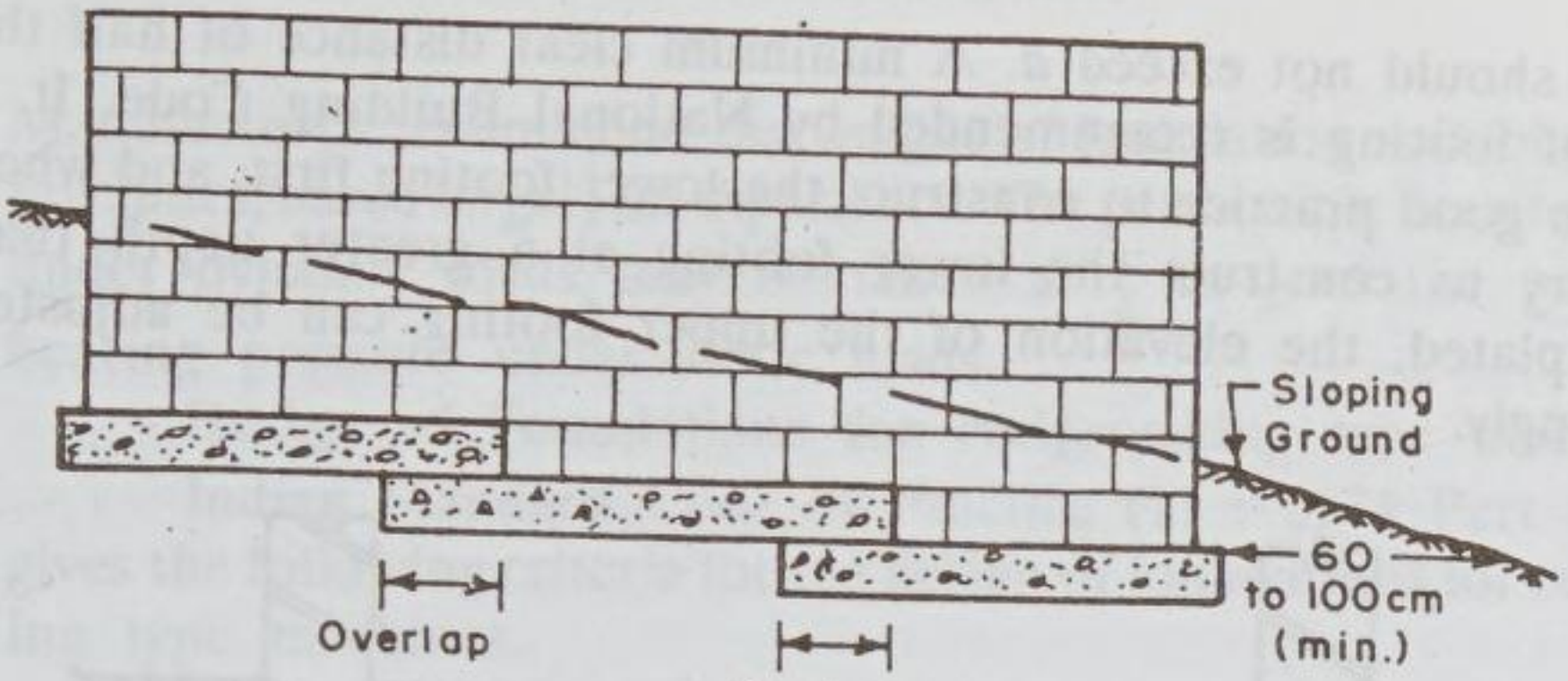


FIG. 3.34. STEPPED FOOTING ON SLOPPING GROUND.

stability analysis is essential. In any case, the distance of the sloping surface from the lower edge of the footing should not be less than 1 m for soils and 60 cm for rock.

Another problem of footing at two different levels is illustrated in Fig. 3.35 where a wall footing at the ground floor adjoint a basement wall. It is common practice to lower the ground floor footings in gradual steps, down to the level of the basement footing as shown. By doing so, the naturai state of the subsoil is considered unaltered.

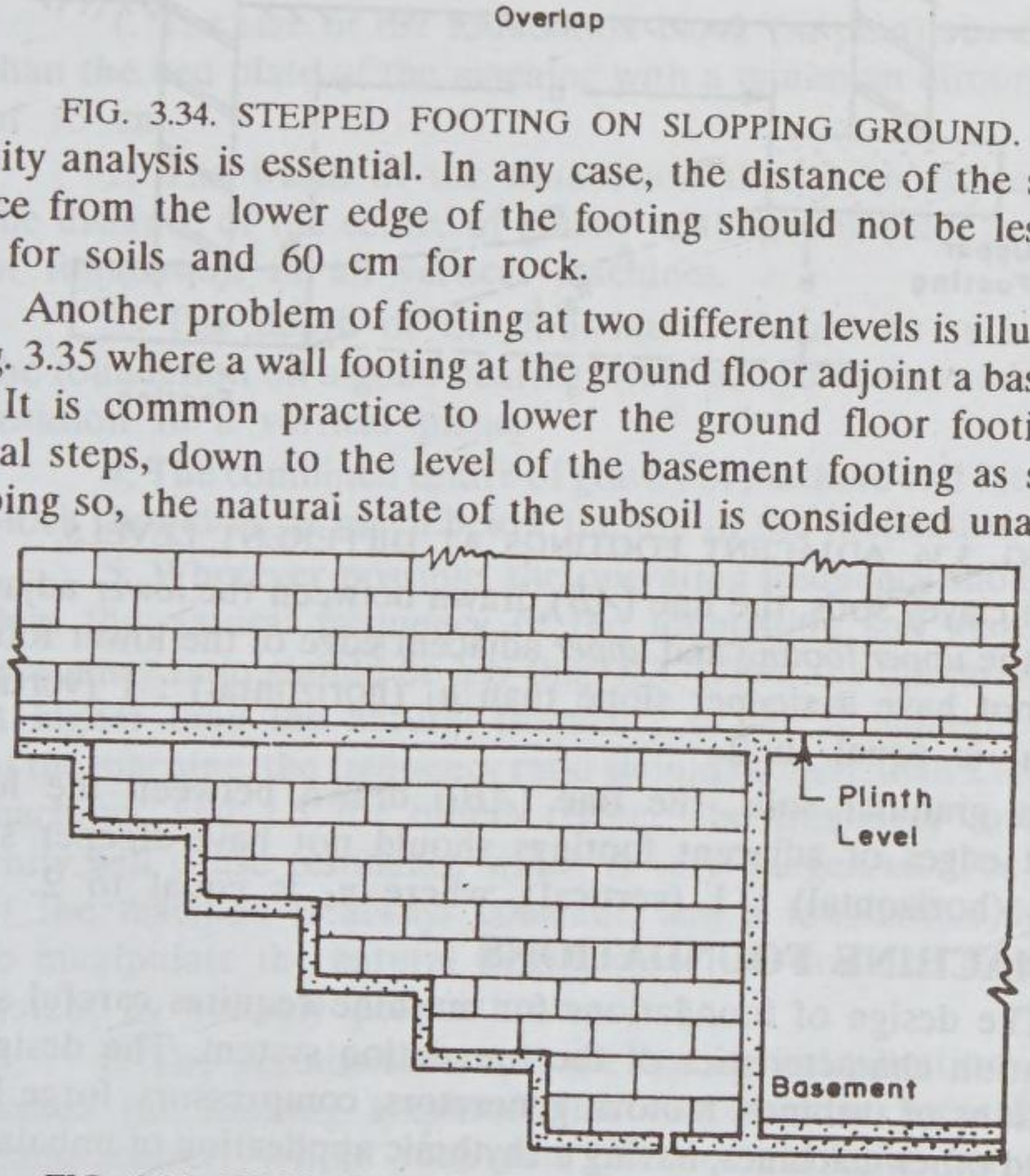


FIG. 3.35. WALL FOOTINGS AT DIFFERENT LEVELS.

3.12. ADJACENT FOOTINGS

Normally, adjacent footings should be placed at the same level. However, when adjacent footings are to be constructed at different levels, the distance between the edges of the footings shall be such as to prevent undesirable overlapping of stresses in soil and disturbance of the soil under the higher footing due to the excavation of the lower footing. The difficulty can be avoided by keeping the difference in footing elevations (b) not greater than one half the clear distance (a) between the footings. However, when footing are founded on

rock, b should not exceed a. A minimum clear distance of half the width of footing is recommended by National Building Code. It is always a good practice to construct the lower footing first, and when necessary to construct the lower footing at a greater depth than contemplated, the elevation of the upper footing can be adjusted accordingly.

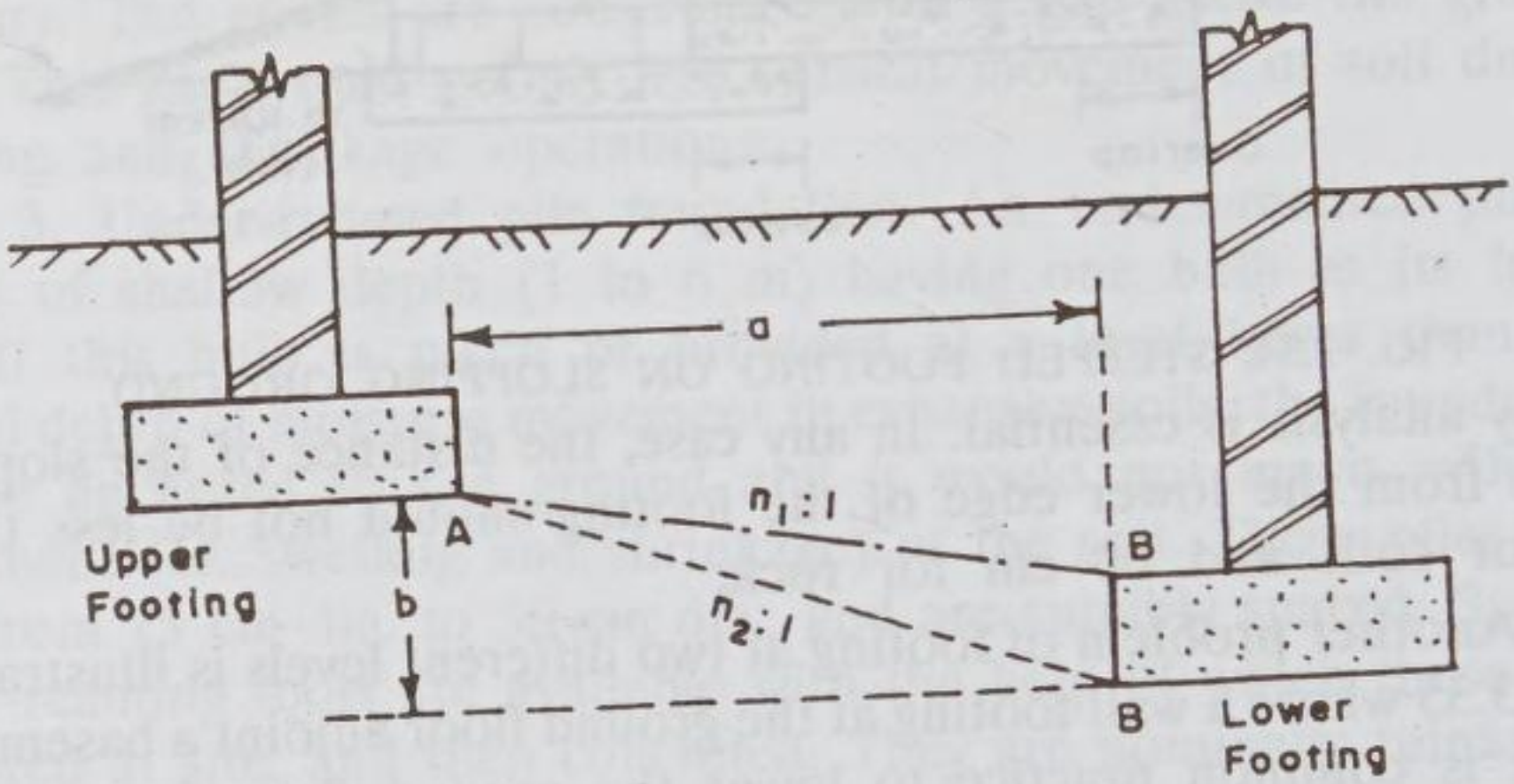


FIG. 3.36. ADJACENT FOOTINGS AT DIFFERENT LEVELS.

In clayey soils, the line (AB) drawn between the *lower* adjacent edge of the *upper footing* and *upper* adjacent edge of the lower footing should not have a steeper slope than n_1 (horizontal) : 1 (vertical), where n_1 is equal to 2.

In granular soils, the line (AB_1) drawn between the lower adjacent edges of adjacent footings should not have steeper slope than n_2 (horizontal) : 1 (vertical), where n_2 is equal to 2.

3.13. MACHINE FOUNDATIONS

The design of foundations for machine requires careful study of vibration characteristics of the foundation system. The design of foundations of turbines, motors, generators, compressors, forge hammers and other machines, having a rhythmic application of unbalanced forces require special knowledge of theory of harmonic vibrations. Inertial forces of rotating elements of machines contribute, besides their *static loads* additional *dynamic loads*. The machinary vibration influences adversely the foundation supporting soil by densifying it which may, in turn, cause differential settlement of the soil and foundation.

Usually mass concrete is used for machine foundations. The excessive vibrations can be eliminated by use of heavy foundations. As a rough guide, the ratio of the weight of foundation to the engine weight may be kept beteween 2.5 to 3.5 in most of the machines.

Manufactures recommend the weight of foundation suitable for their machines, based largely on experience. The permissible bearing pressure under dynamic loads may be taken as $\frac{1}{2}$ to $\frac{1}{4}$ of the permissible bearing pressure under static loads.

Design of Foundations for reciprocating type machines.

Indian Standard Code of Practice (IS : 2974 Part I : 1964) gives the following criteria for the design of foundations for reciprocating type machines.

1. The size of the foundation block (in plan) should be larger than the bed plate of the machine with a minimum alround clearance of 15 cm.

2. The width of the foundation should be at least equal to the distance of the centre of gravity of the crank shaft to the bottom of foundation in all vertical machines.

3. The depth of the foundation should be such as to rest the foundation on a good bearing strata and to ensure stability against rotation in a vertical plane.

4. The combined centre of gravity of machine and the foundation block should be as much below the top of the foundation as possible.

5. Wherever possible, the operating frequency should be lower than the natural frequency of the foundation soil system and the frequency ratio should be *less* than 0.5. When the operating frequency is higher than the natural frequency of the foundation soil-system of the machine, the frequency ratio should be *more* than 2 for important machines and 1.5 for others (Note. The frequency ratio equal to unity will cause resonance which is very dangerous. The frequendcy of the machine is always constant, and a foundation designer has to manipulate the natural frequency of the machine foundation-soil system by suitably proportioning it).

6. The permissible *amplitude* for vertical vibrations should not exceed the limiting amplitude for the machine prescribed by the manufacturer. Where such data are not available, for preliminary design and for relatively unimportant structures, the limiting amplitude may be determined from Fig. 3.37.

Design of Foundations for Impact type machines.

The design requirements of the impact type machines such as drop and forge hammers, are different than those of the reciprocating type machines described above. Indian Standard Code (IS : 2974: Part II : 1966) covers the design requirements for the foundations of these heavy impact type machines. Fig. 3.38 shows some typical sections of the foundations for these machines.

Definitions : (*i*) *Anvil* : Anvil is a base block for a hammer on which material is forged into shape by repeated striking of the

tup. (*ii*) *Tup* : Tup is a weighted block which strikes on material being forged on the anvil. (*iii*) *Foundation block* : It is a mass of reinforced concrete on which the anvil rests. (*iv*) *Protective cushioning Layer* (Joint J_1) : It is an elastic cushioning of suitable material and thickness provided between the anvil and the foundation block in order to prevent bouncing of anvil and creation of large impact stress and consequent damage to the top surface of the concrete in the foundation block. (*v*) *Foundation support* (Joint J_2)) : It is support for resting the foundations block. The block may be directly on ground or on a resilient mounting such as timber sleepers, spring cork layer etc. The block may also be supported on pile foundation.

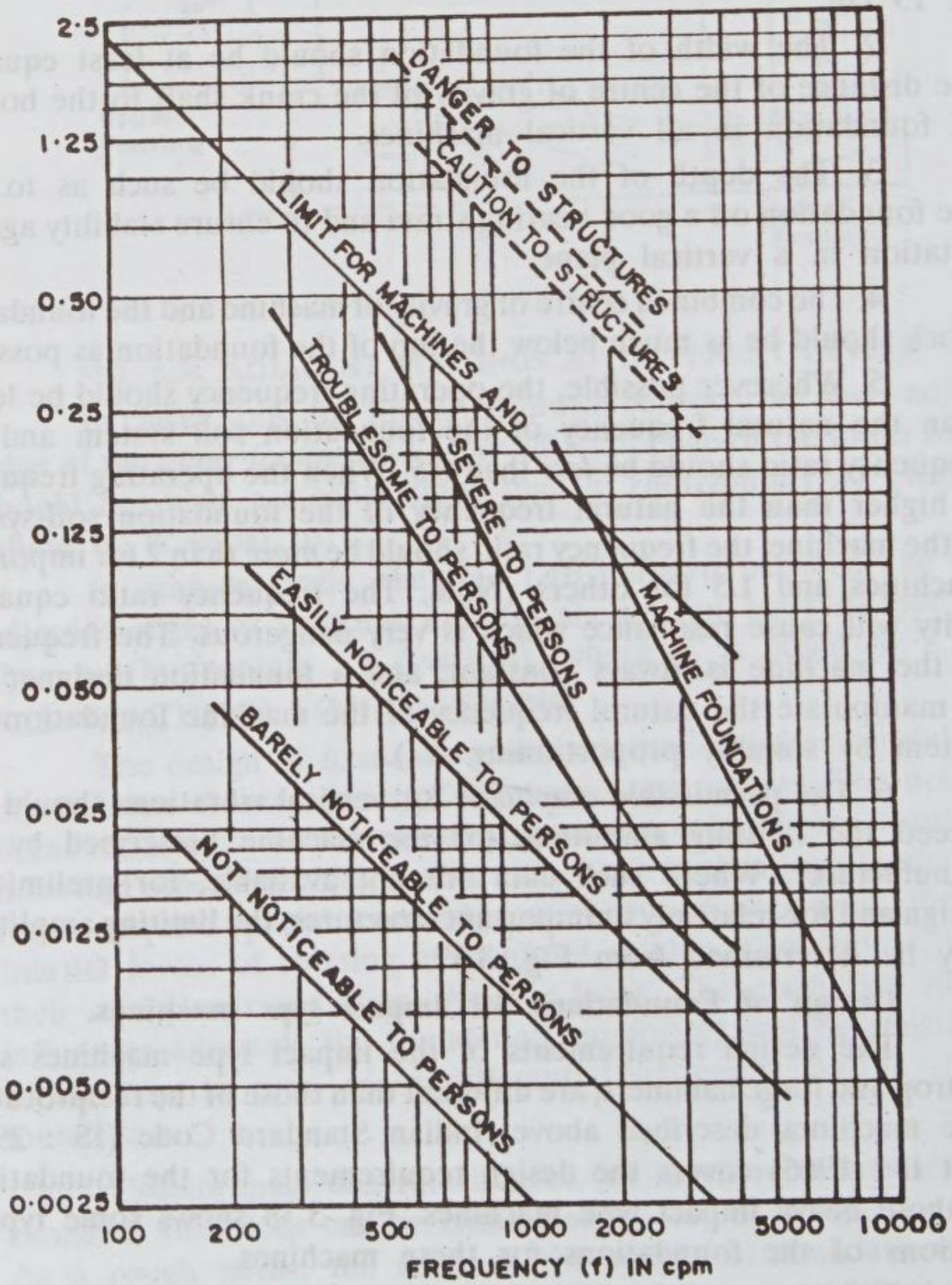
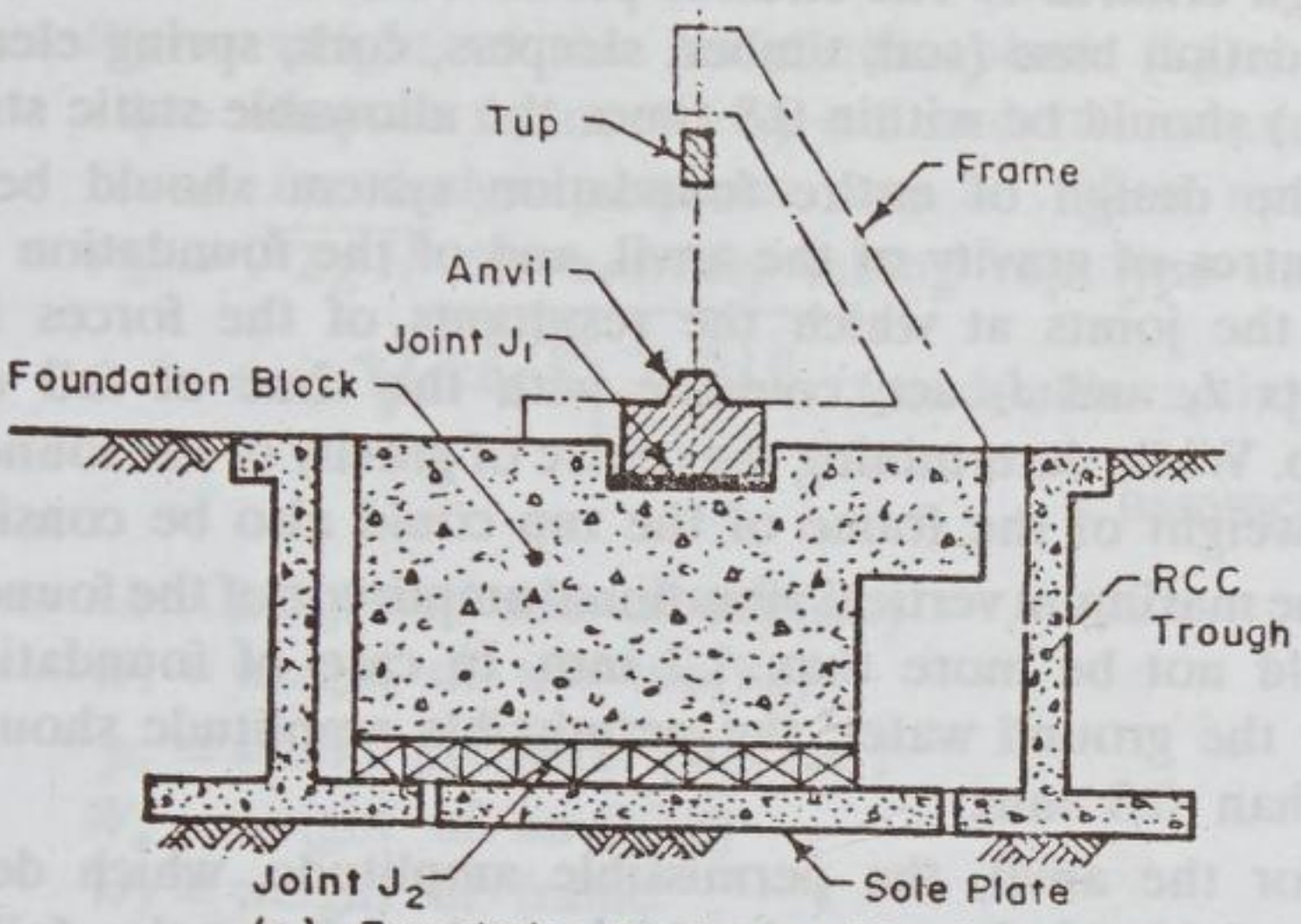
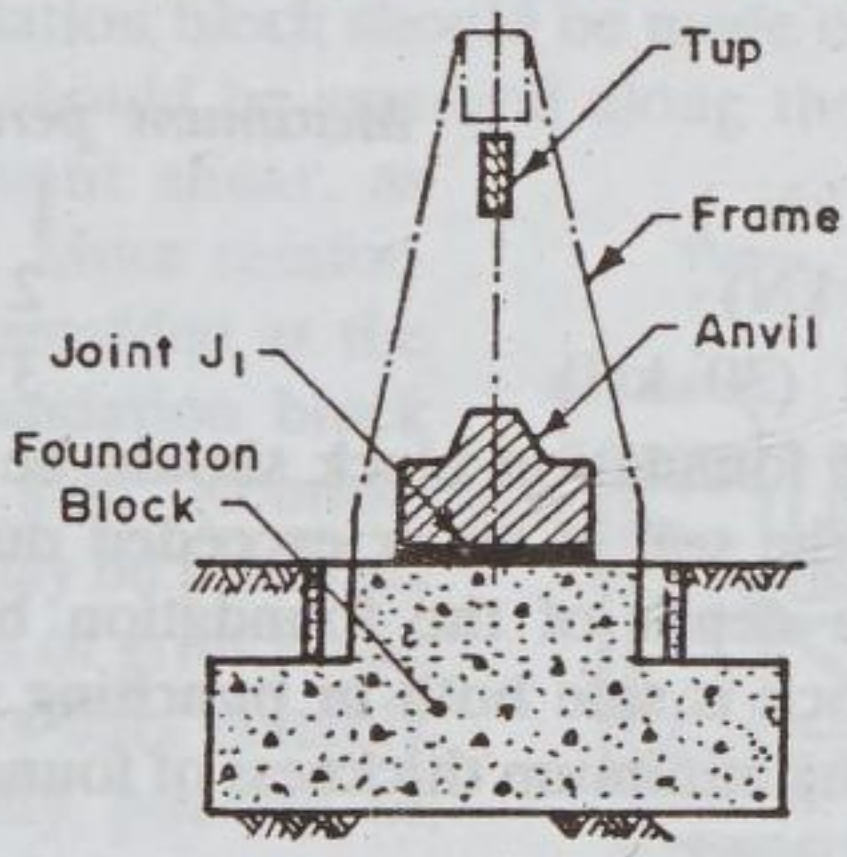


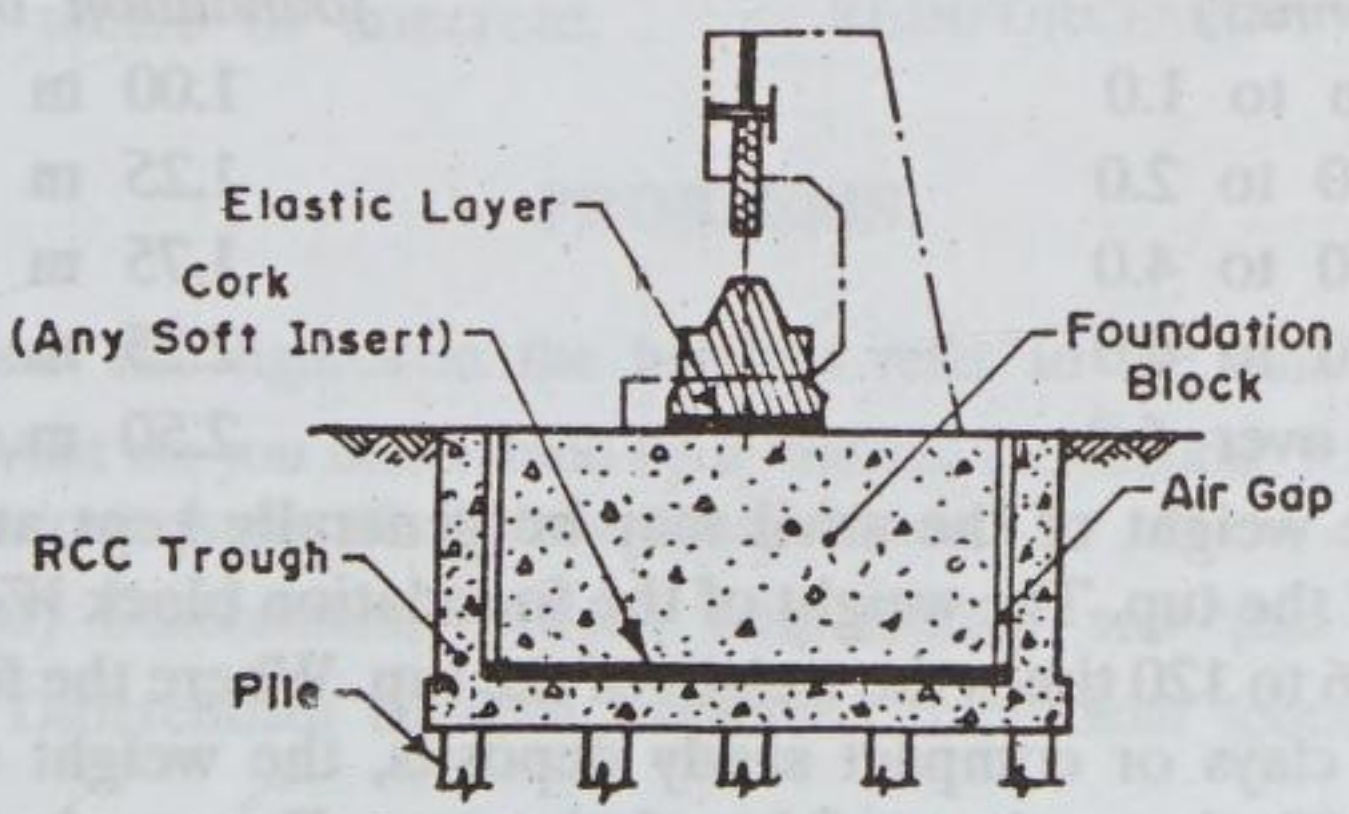
FIG. 3.37. LIMITING AMPLITUDE FOR VERTICAL VIBRATIONS.



(a) Foundation Having an Elastic Support



(b) Foundation Resting Directly on Soil



(c) Foundation Resting on Piles

FIG. 3.38. DIFFERENT TYPES OF FOUNDATION SUPPORTS FOR IMPACT TYPE MACHINE.

Design criteria 1. The stresses produced at the time of impact in the foundation base (soil, timber, sleepers, cork, spring elements, or piles etc.) should be within 0.8 times the allowable static stresses.

2. The design of entire foundation system should be such that the centres of gravity of the anvil, and of the foundation block, as well as the joints at which the resultants of the forces in the elastic joints J_1 and J_2 act, coincide with the time of fall of the hammer tup. While determining the centre of gravity of the foundation block, the weight of the frame of the tup could also be considered.

3. The maximum vertical vibrational amplitude of the foundation block should not be more than 1.2 mm. In case of foundation on sand below the ground water, the permissible amplitude should not be more than 0.8 mm.

4. For the anvil, the permissible amplitude, which depends upon the weight of the tup should be taken from the following table:

Weight of tup	*Maximum permissible amplitude*
Upto 1t (10 kN)	1 mm
2t (20 kN)	2 mm
More than 3t (30 kN)	3 to 4 mm

5. The area of foundation block should be such that the safe loading intensity of the soil is never exceeded during the operation of the hammer. The depth of the foundation block should be so designed that the block is safe both in punching shear and bending. However, the following minimum thickness of foundation block should be provided:

Weight of tup (tonnes)	*Minimum depth of foundation block*
up to 1.0	1.00 m
1.0 to 2.0	1.25 m
2.0 to 4.0	1.75 m
4.0 to 6.0	2.25 m
over 6.0	2.50 m.

6. The weight of the anvil may be generally kept at 25 times the weight of the tup. The weight of the foundation block W_b generally varies from 66 to 120 times the weight of the tup. Where the foundation rest on stiff clays or compact sandy deposits, the weight should be from 75 to 80 times the weight of the tup. For moderately firm to soft clays and for medium dense to loose sandy deposits, the weight of the block should be from 90 to 120 times the weight of the tup.

The approximate weight of the foundation block may also be determined from the following formula :

$$W_b = 0.08\,(1 + I_f)\,V_{tb}\,W_t - (W_1 + W_f) \qquad ...(3.17)$$

where W_b = weight of the foundation block (kg)

I_f = impact factor $0 < I_f < 1$ and its average value for design purposes may be taken upto 0.6

$V_{tb} = \sqrt{2gH}$ for a freely falling tup type hammer

$$= 0.65\sqrt{\frac{2g\,(W_t + p_s)\,h}{W_s}}$$ for double acting steam hammers ...(3.18)

h = height of fall of tup(cm)

W_t = weight of the tup(kg)

p_s = steam pressure (kg/cm^2)

W_a = weight of anvil(kg)

W_f = weight of frame.

7. The foundation block should be made of reinforced concrete and reinforcement should be arranged along the three axes and also diagonally to prevent shear, as shown in Fig. 3.39. More reinforcement should be provided at the top side of the foundation block than at the other side. Reinforcement at the top may be provided in the form of layers of grills made of 16 mm diameters bars suitably spaced to allow easy pouring of concrete. The reinforcement provided should be at least 25 kg per cubic metre of concrete.

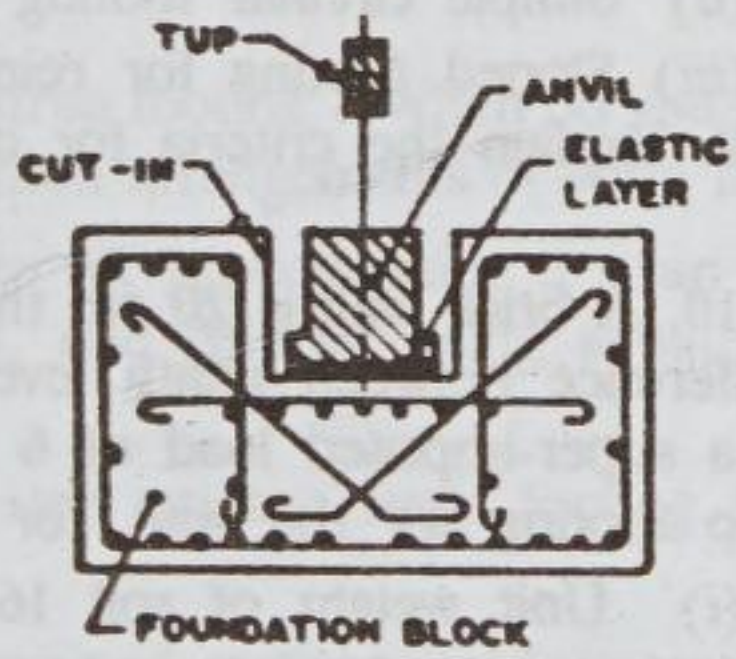


FIG. 3.39. TYPICAL REINFORCEMENT DETAILS.

PROBLEMS

(**Note.** The figures in the brackets refer to the data in SI Units.)

1. What do you understand by a 'shallow foundation' ? Draw sketches to show various types of shallow foundations.

2. (*a*) Differentiate between 'strip footing' and 'pad footing'.

(*b*) Differentiate between 'combined trapezoidal footing' and 'strap footing.'

3. (*a*) Explain what are the criteria for determining minimum depth of shallow foundations.

(*b*) If the safe bearing pressure of the soil, and the angle of spread of the load from the wall base to the outer edge of the ground bearing

is given, how do you fix the minimum depth of foundation ?

4. A wall of width T requires a footing width B to transmit the load safely to the foundation. If the angle of spread of the load from wall base to the outer side of the ground bearing are not to exceed $n:1$ for masonry and $n_1:1$ for foundation concrete, show that minimum depth of foundation required to achieve the above is given by

$$D_{min} = \frac{1}{2n}\left[(B - T) - 2\,d(n_1 - n)\right]$$

What will be the form of the above expression if $n = \frac{1}{2}$ and $n_1 = 1$

5. Derive an expression for the depth of concrete block required for a strip footing of a wall.

6. Discuss in brief the method of designing the components of a strip footing.

7. Explain how do you design the 'stepped pad footing' for a masonry square column. How will the design be changed if it is a rectangular column?

8. Draw typical sketches to show the following :

(*i*) Simple square footing for reinforced concrete column.

(*ii*) Simple circular footing for reinforced concrete circular column.

(*iii*) S!oped footing for reinforced concrete rectangular column.

9. Explain the criteria for designing footing for eccentrically loaded wall.

10. A brick wall is 20 cm thick and 3 m high above the plinth level. The difference between plinth level and ground level is 0.6 m. The wall carries a super-imposed load of 6 tonnes (60 kN) per metre run. Design the strip footing for the wall, for the following data :

(*i*) Unit weight of soil 1650 kg/m^3 (16.5 kN/m^3)

(*ii*) Angle of repose of soil 24°

(*iii*) Unit weight of masonry 2000 kg/m^3 (20 kN/m^3)

(*iv*) Unit weight of foundation concrete (cement concrete) 2300 kg/m^3 (23 kN/m^3).

(*v*) Modulus of rupture for the concrete (1 : 3 : 6) : 35 t/m^2 (350 kN/m^2)

(*vi*) Safe bearing capacity of soil : 9 t/m^2 (90 kN/m^2)

Sketch the foundation section.

11. Design the isolated stepped footing for a brick pillar, 30 cm × 30 cm, carrying a super-imposed load of 25 t (250 kN) at its top. The height of the column above ground level is 3.5 m. The brick masonry weighs 1950 kg/m^3 (19.5 kN/m^3) while lime concrete to be used in the base weighs 2100 kg/m^3(21 kN/m^3). The soil has angle of repose of 30°}, unit weight of 1700 kg/m^3 (17 kN/m^3) and safe bearing capacity of 16 t/m^2 (160 kN/m^2). The foundation concrete has a modulus of rupture equal to 15 t/m^2 (150 kN/m^2).

12. What do you understand by grillage foundation ? Draw a neat sketch of steel grillage foundation for a steel stanchion. Explain the method of construction.

13. Sketch typical timber grillage foundations for the following :

(*i*) masonry wall.

(*ii*) wooden post.

14. What is meant by a 'combined footing' ? When do you adopt it ? What modification will you make if one of the columns lies just at the edge of the adjacent property ?

15. Explain in detail the procedure for proportioning a rectangular combined footings for two columns carrying unequal loads. The distance between the columns is given.

16. Draw a typical sketch for combined steel grillage rectangular footing for two steel stanchions.

17. Explain in detail the procedure for proportioning a trapezoidal combined footings for two columns carrying unequal loads. The distance between the columns is given.

18. Draw a typical sketch for combined steel grillage trapezoidal footing for two steel stanchions.

19. What do you understand by a strap footing ? When do you provide this ? Draw typical sketches showing common arrangements of strap footings.

20. Explain in detail the procedure for proportioning a strap footing for two columns carrying unequal loads, one column being situated hear a property line.

21. Draw a typical sketch for a steel grillage strap footing for two stanchions.

22. Find the dimensions of a combined rectangular footing for two columns carrying load of 40 t (400 kN) and 60 t (600 kN) respectively. The columns are spaced 3 m centre to centre. The safe bearing capacity of the soil is 10 t/m^2 (100 kN/m^2).

23. Proportion a combined footing for two R.C. columns *A* and *B* separated by a distance of 4 m centre to centre.:

Column *A* is 50 cm square and carries a load of 120 t (1200 kN)

Column *B* is 60 cm square and carries a load of 160 t (1600 kN)

The safe bearing capacity of the soil is 20 t/m^2 (200 kN/m^2).

24. Design a trapezoidal combined footing for the following requirements:

Column *A* = 40 cm × 40 cm.

Column *B* = 60 cm × 60 cm

Axial laod on column *A* = 40 t (400 kN).

Axial load on column *B* = 60 t (600 kN).

Distance between centres of columns = 2.5 m.

Safe bearing capacity of soil $= 20\ t/m^2\ (200\ kN/m^2)$.

The footing is not to project more than 0.4 m beyond the outer faces of the columns.

25. What do you understand by raft foundation ? When do you prefer this?

Explain with the help of sketches common types (or forms) of raft foundation.

26. What are the problems of foundations on black cotton soils ? What points should be kept in mind while designing foundations in such soild ?

27. Draw typical sketches of sections of shallow foundations on expansive soils. Explain the functions of special provisions made in each case.

28. Write notes, explaining design criterion for the following :

(*i*) Stepped footing for sloping ground.

(*ii*) Adjacent strip footings at different levels.

29. Explain in brief general rules for the design of foundation for reciprocating engines.

30. Explain in brief the design criteria for foundation for impact type machines.

4

Foundations-3 : Deep Foundations

4.1. INTRODUCTION

Deep foundations are those in which the depth of the foundation is very large in comparison to its width. Deep foundations are not constructed by ordinary methods of open pit excavations. Deep foundations are of the following types :

1. Pile foundation.
2. Pier foundation
3. Caisson or well foundation.

Out of these, pile foundation is more commonly used in building construction. Following are the situations in which a pile foundation is preferred:

1. The load of the super structure is heavy and its distribution is uneven.
2. The top soil has poor bearing capacity.
3. The subsoil water level is high so that pumping of water from the open trenches for the shallow foundations is difficult and uneconomical.
4. There is large fluctuations in subsoil water level.
5. If deep strip foundation is attempted, timbering of sides is difficult to maintain or retain the soil of sides of the trench.
6. The structure is situated on the sea shore or river bed, where there is danger of scouring action of water.
7. Canal or deep drainage lines exist near the foundations
8. The top soil is of expansive nature.

4.2. TYPES OF PILES

The use of piles as a foundation can be traced since olden times. The art of driving piles was well established in Roman times

and the details of such foundations were recorded by Vitruvious in 59 A.D. Today, pile foundation is much more common than any other type of deep foundation. Modern pile driving started with the first steam pile drivers, invented by Nasmyth in 1845.

Piles may be classified as follows :

(*a*) **Classification based on function**

Based on the function or the use, piles may be classified as : (1) end bearing pile (2) friction pile (3) compaction pile (4) tension pile or uplift pile (5) anchor pile (6) fender pile and dolphins (7) batter pile (8) sheet pile.

End bearing piles are used to transfer load through water or soft-soil to a suitable bearing stratum [Fig 4.1(*a*)].

Friction piles are used to transfer loads to a depth of a friction load carrying material by means of skin friction along the length of piles [Fig. 4.1(*b*)].

Compaction piles are used to compact loose granular soils,

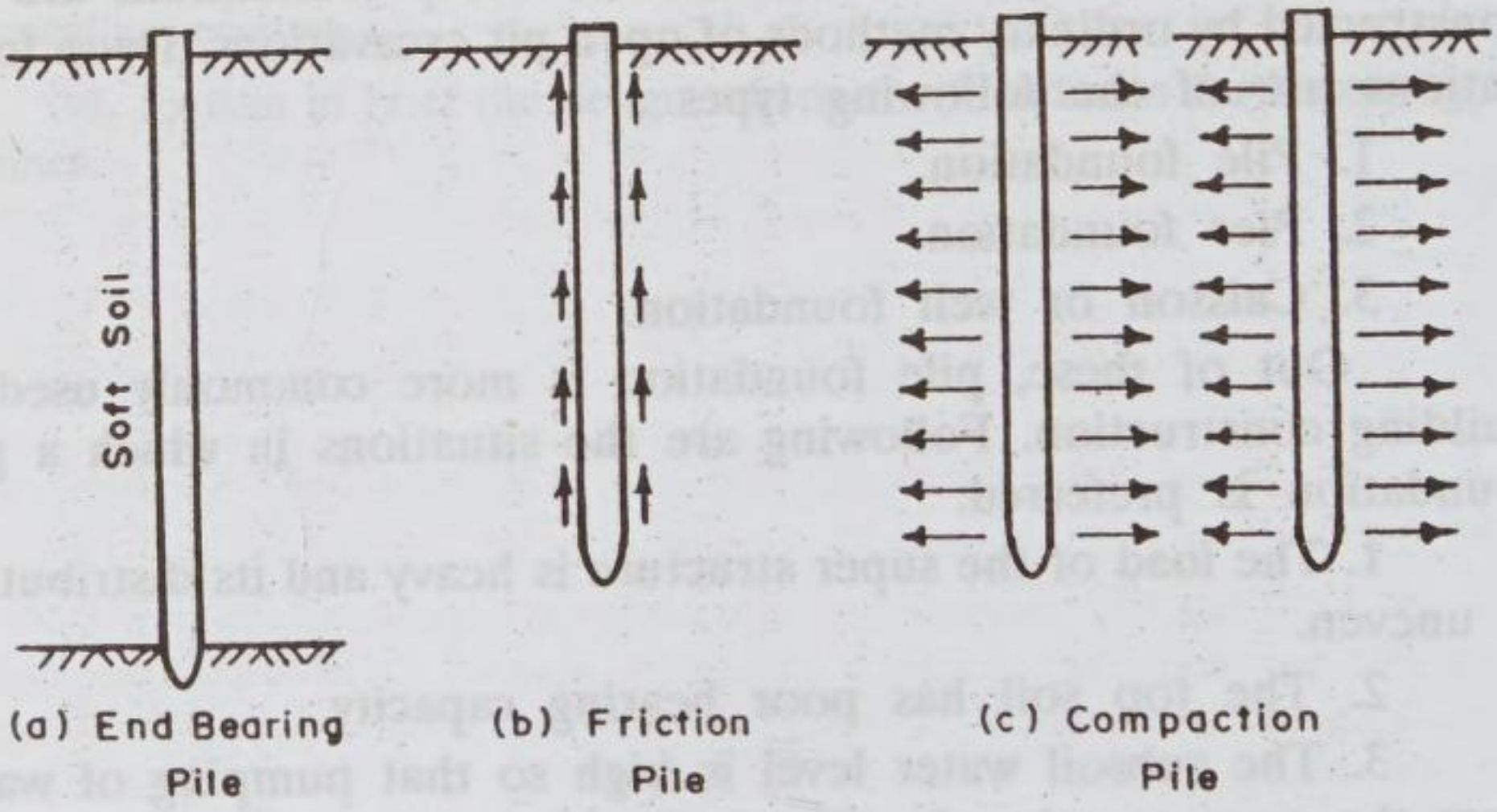


FIG. 4.1. CLASSIFICATION OF PILES BASED ON FUNCTION.

thus increasing their bearing capacity. The compaction piles themselves do not carry any load. Hence they may be of weaker material–sometimes of sand only. The pile tube, driven to compact the soil, is gradually taken out and sand is filled in its place thus forming a 'sand pile' [Fig. 4.1 (*c*)].

Tension or uplift piles anchor down the structures subjected to uplift due to hydrostatic pressure or due to over-turning moment [Fig. 4.1 (*d*)].

Anchor piles provide anchorage against horizontal pull from sheet piling or other pulling forces [Fig. 4.1 (*e*)].

Fender piles and dolphins are used to protect water from structures against impact from ships or other floating objects.

Sheet piles are commonly used as bulkheads, or as impervious cut off to reduce seepage and uplift under hydraulic structures. The batter piles are used to resist large horizontal or inclined forces [Fig. 4.1 (*f*)].

(*b*) Classification based on materials and composition

1. ***Concrete piles*** :
   - (*a*) Pre-cast
   - (*b*) Cast-in-situ
     - (*i*) Driven piles : Cased or uncased
     - (*ii*) Bored piles : Pressure piles, under–reamed piles and bored compaction piles.
2. ***Timber piles***
3. ***Steel piles***
   - (*a*) H-pile
   - (*b*) Pipe pile
   - (*c*) Sheet pile
4. ***Composite piles***
   - (*a*) Concrete and timber
   - (*b*) Concrete and steel.

The *precast concrete piles* are generally used for a maximum design load of about 80 tonnes (800 kN), except for large pre-stressed piles. They must be reinforced to withstand handling stresses. They require space for casting and storage, more time to set and cure before installation and heavy equipment for *handling* and driving. They also incur large cost in cutting for extra length or adding more length.

The *cast-in-situ* concrete piles are generally used for a maximum design load of 75 tonnes (750 kN) except for compacted pedestal piles. They are installed by pre-excavation, thus eliminating vibration

due to driving and the handling stresses. Cast-in-situ piles may be classified into two classes : (*i*) *driven piles* (cased or uncased) and *bored piles* (pressure piles, pedestal piles and under-reamed piles). A variety of cast-in-situ piles are in use, each bearing the name of the manufacturer.

Under-reamed pile is a special type of bored pile having an increased diameter or bulb at some point in its length, to anchor the foundation in expansive soil subjected to alternate expansion and contraction.

Concrete filled steel piles and *steel H-piles* are used as long piles with high bearing capacity. They are rarely used unless they reach a stratum of exceptionally high supporting capacity, since their cost is very high. *Timber piles* have small bearing capacity, and are not permanent unless treated. They are prone to damage by hard driving, and should not be driven through hard stratum or boulders. *Composite piles* are suitable where the upper part of a pile is to project above water table. Such a pile consists of a lower portion of untreated timber and an upper portion of concrete. In other types of composite piles, steel piles are attached to the lower end of cast-in place concrete piles. This type is used in case where the required length of pile is greater than that available for the cast-in-place type.

4.3. CASED CAST-IN-SITU CONCRETE PILES

Cased cast-in-situ piles are suitable in practically all ground conditions. The shell is driven into intimate contact with the surrounding soil and remains in place to maintain driving resistance and protect the concrete filling during the placing of other adjacent piles and during the critical setting period. Cased piles can be easily cut or extended to meet variations in shell length. One of the main advantages is that it is subject to internal inspection after it is driven. The following are the common types of cased cast-in-situ concrete piles :

(*i*) Raymond standard pile and step-taper pile
(*ii*) Mc-Arthur cased pile
(*iii*) Union metal monotube pile
(*iv*) Swage pile
(*v*) Western button bottom pile.

1. Raymond piles.

In 1897, A.A. Raymond patented the Raymond pile system and was first to develop a practical, economical way of placing cast-in-situ concrete piles. Two types of Raymond piles are in use (*a*) Raymond standard concrete pile and (*b*) Raymond step-taper concrete pile.

(a) Raymond Standard Concrete pile.

The Raymond Standard pile is used primarily as a friction pile since its uniform heavy taper of 1 in 30 usually results in shorter piles for equal driving resistance or higher driving resistance for equal lengths, than piles of lesser or no taper. The lengths of piles vary from 6 to 12 m. The diameter of piles vary from 40 to 60 cm at the top and 20 to 30 cm at the bottom. The pile consists of a thin *corrugated steel shell* closed at the bottom. The steel shell is reinforced with spirally wound hard drawn wire on 8 cm pitch. The shell is closed at the bottom with a steel boot. The shell is driven into the ground with a collapsible steel *mandrel* or *core* in it having the same taper. When the pile is driven to the desired depth the mandrel is mechanically collapsed and withdrawn, leaving the shell inside the ground. The shell is inspected internally by using the light from a mirror or flash light or drop light. The shell is gradually filled with concrete up to the top.

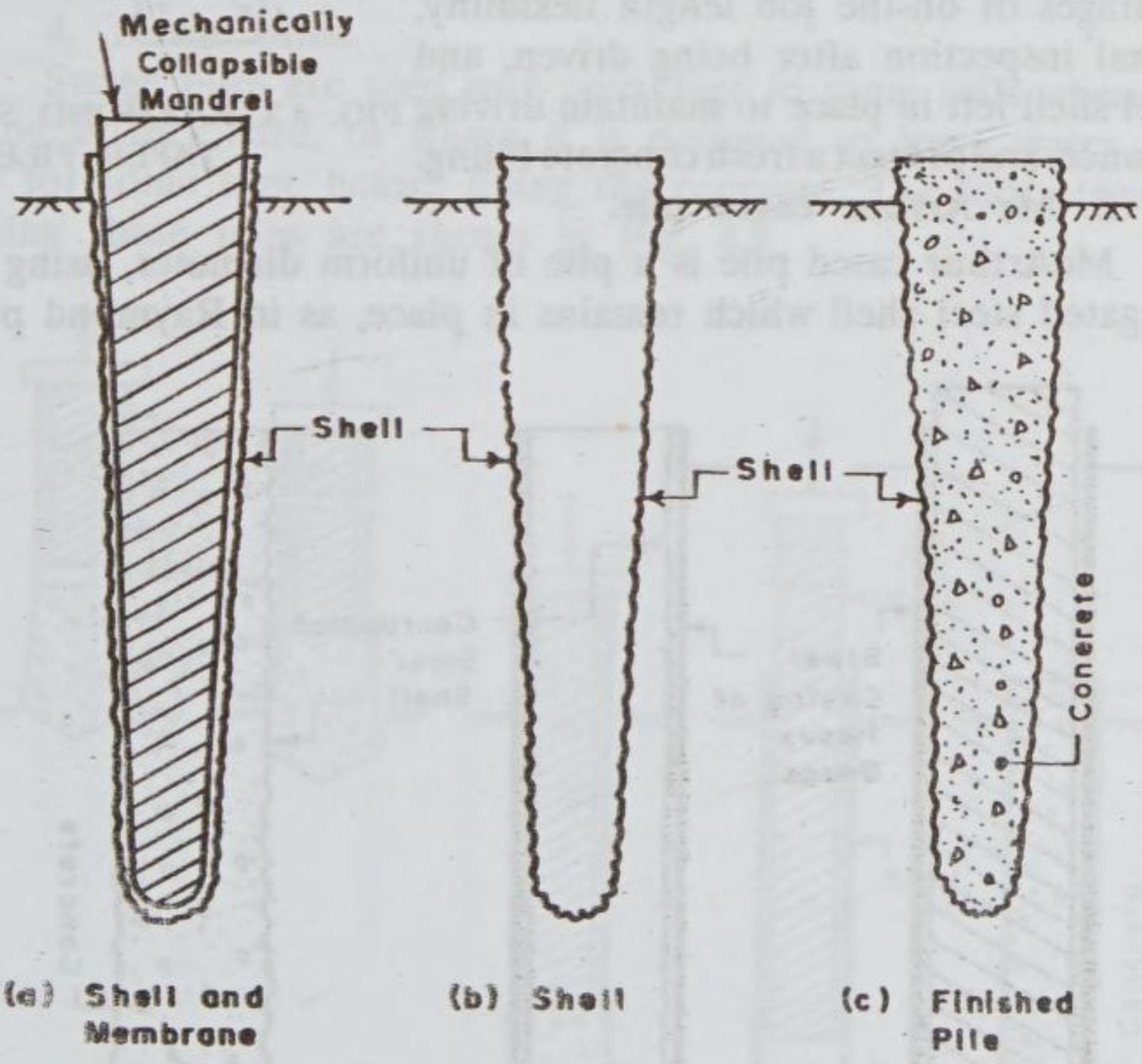


FIG. 4.2. STAGES IN RAYMOND STANDARD PILE CONSTRUCTION.

(b) Raymond step-taper concrete pile .

This type of mandrel-driven pile is used either as an end-bearing or friction pile, and can be driven in any type of soil. The pile

uses shell sections driven in different lengths. The bottom most section of shell is made of heavier gauge, and is closed by flat steel plate welded to the boot ring. The joints between sections of shell are screw-connected. The shells are driven with a rigid internal steel mandrel or core which is stepped to conform with the shell sections used. The heavy rugged core provides a high degree of penetration and efficiently transmits hammer energy to the bearing strata. The pile diameter increases in steps at the rate of 2.5 cm for each successive shell section.These can be drawn upto a maximum depth of about 36 m, using 20 cm tip. The method of forming the pile is the same as that for the standard pile. The pile has the advantages of on-the job length flexibility, internal inspection after being driven, and a steel shell left in place to maintain driving resistance, and protect a fresh concrete filling.

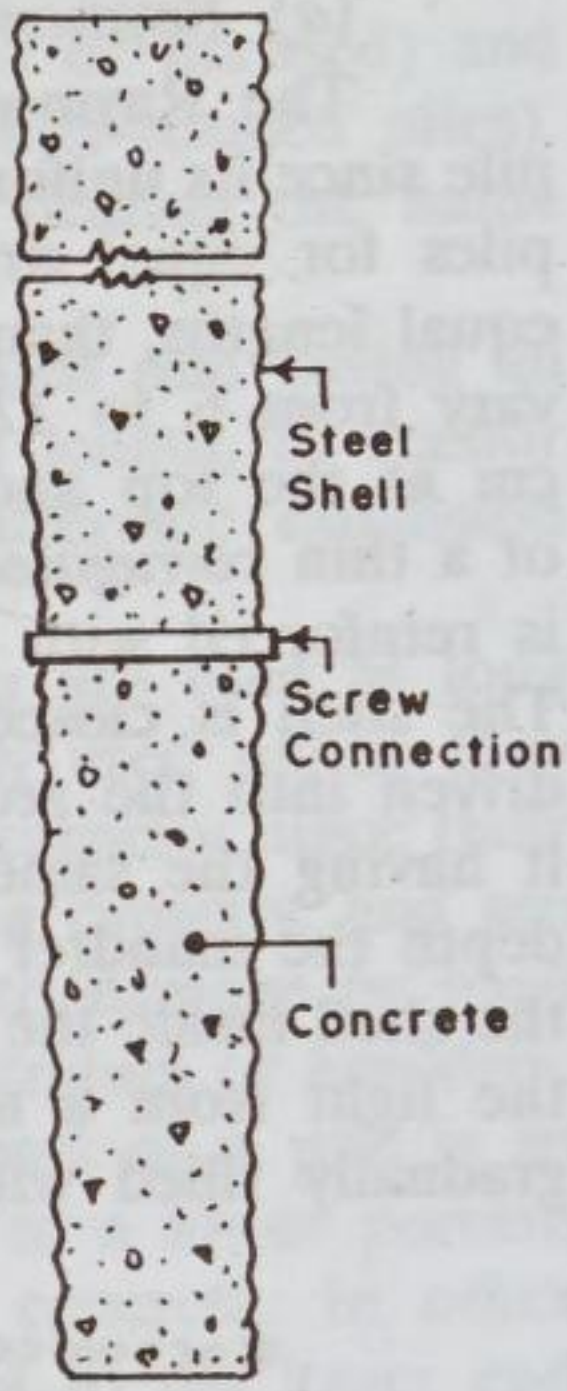


FIG. 4.3. RAYMOND STEP-TAPER PILE.

2. Mc Arthur cased pile.

McArthur cased pile is a pile of uniform diameter, using the corrugated steel shell which remains in place, as in Raymond piles.

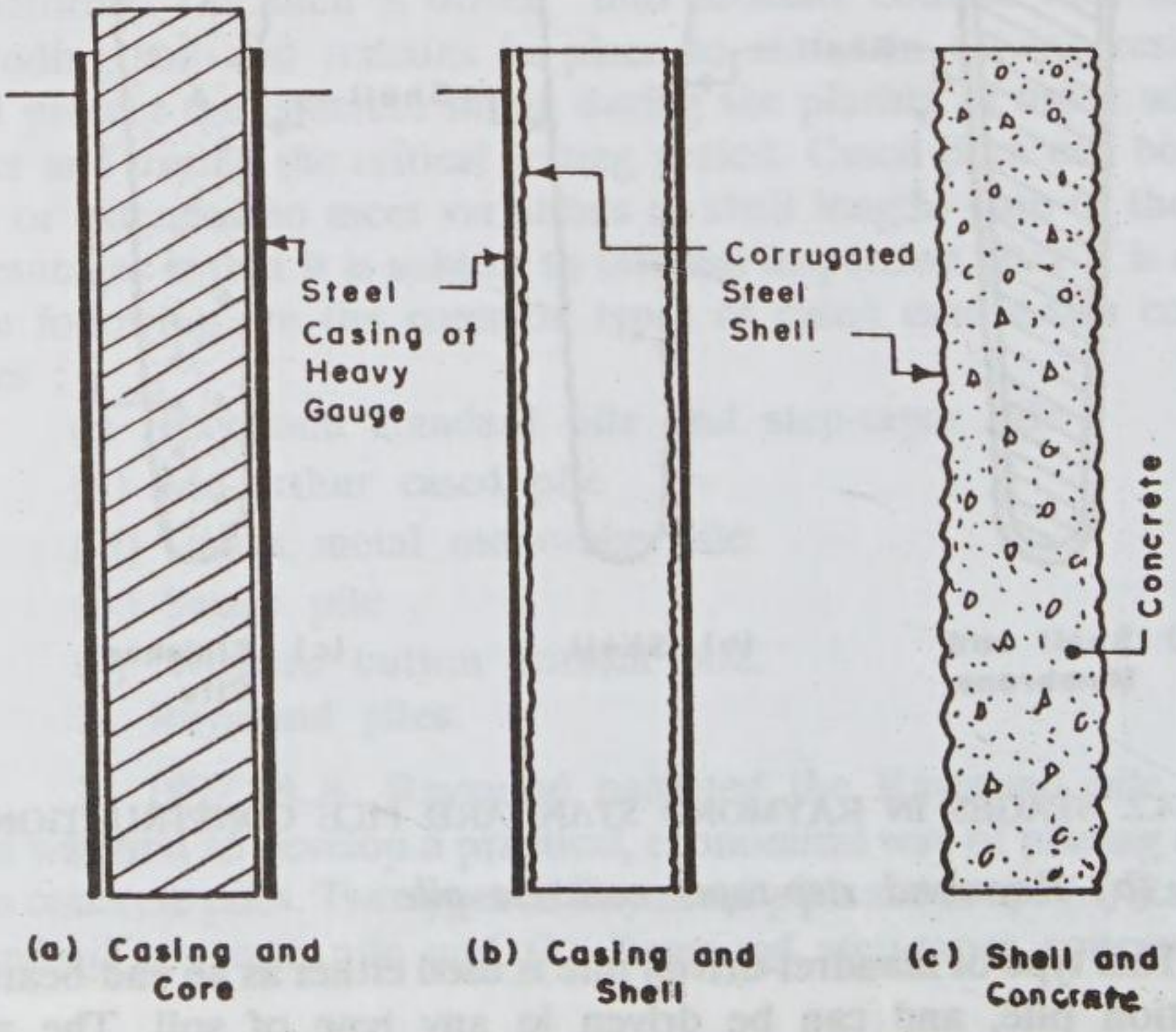


FIG. 4.4 STAGES IN Mc-ARTHUR CASED PILE.

However the driving of the pile uses an additional steel casing of heavy gauge. The heavy steel casing with a central core is driven into the ground as shown in Fig. 4.4 (*a*). After reaching the desired depth, the central core is withdrawn, and a corrugated shell is placed in the casing [Fig. 4.4 (*b*)]. Finally, concrete is placed in the shell, by gradually compacting it, and withdrawing the steel casing. The completed pile, shown in Fig. 4.4 (*c*) contains concrete core and the outer corrugated shell.

3. Union metal monotube pile.

Monotube piles uses tapered fluted steel shell without mandrel, and are suitable for a wide variety of soil conditions, from end-bearing to friction-load-carrying soils. The shells provide rigidity, and are watertight. The pile shells are driven to the required depth, and they are inspected after driving. The stiffness of the shell against crushing from adjacent piles is very good. Shells may be driven with hammer of comparable size to those used for wood piles. The shell, after inspection, is filled with concrete, and the excess length of the shell, if any, is cut.

4. Swage piles.

Swage piles are used with advantage in some soils where the driving is very hard, or where it is designed to leave water tight shell for some time before filling the concrete. The four stages of forming these piles are shown in Fig. 4.5.

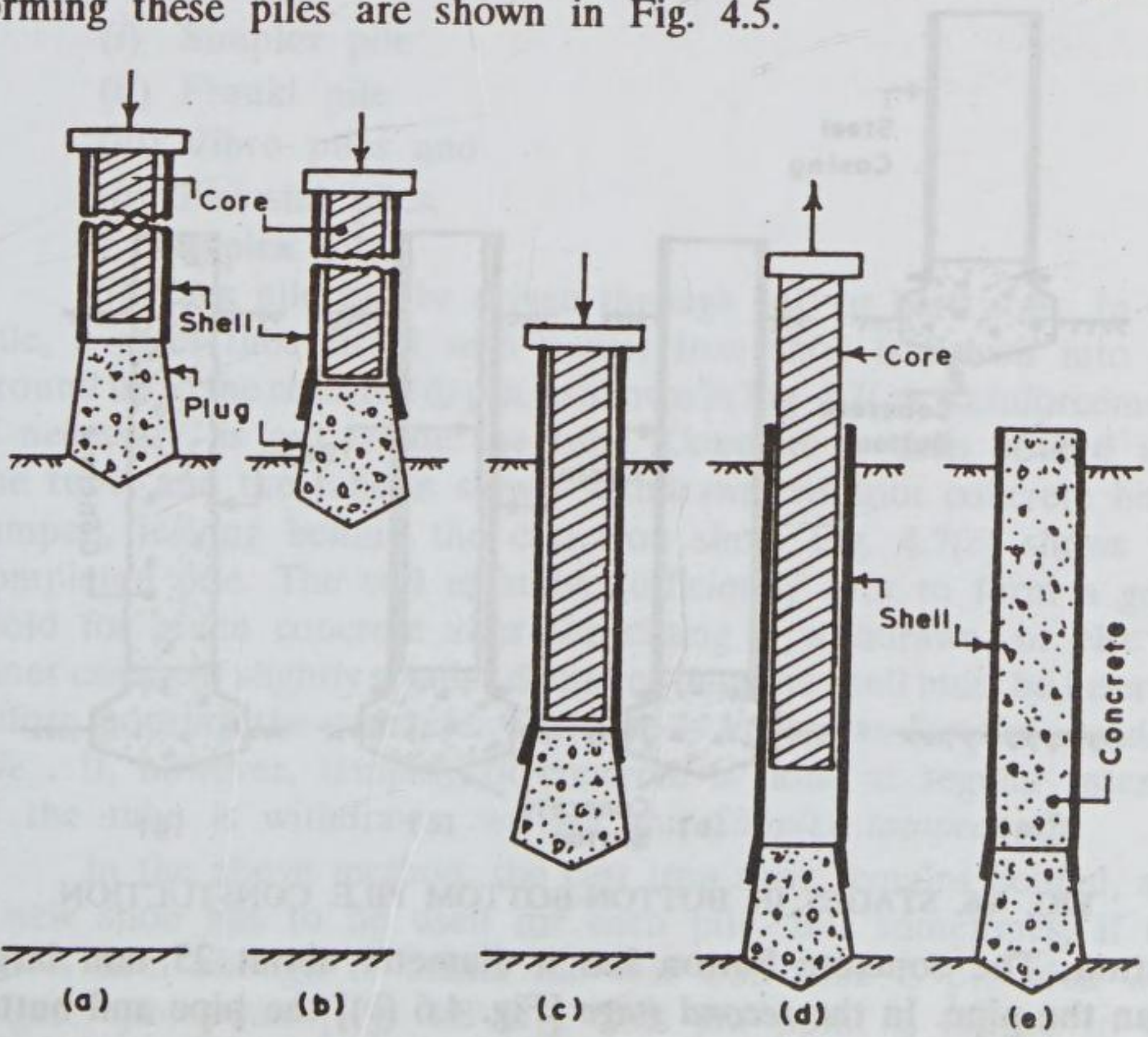


FIG. 4.5. STAGES IN SWAGE PILE CONSTURCTION

In the *first stage* [Fig. 4.5 (*a*)] a thin steel pile (known as shell) is placed on a *precast* concrete plug, and a steel core, which is not long enough to reach the plug is inserted in the shell. In the *second stage* [Fig. 4.5(*b*)] as the pipe is driven over the plug until the core reaches the plug, the pipe is *swaged out* by the taper of the plug, thus forming a water tight joint. In the *third stage* [Fig. 4.5 (*c*)] the pipe is driven to a specified depth. The driving force is practically all exerted by the core on the plug and the pipe pulled down rather than driven. In the fourth stage [Fig. 4.5 (*d*)] after the pipe has reached the desired depth, the core is removed, and the pipe left open until it is desired to fill it. In the *final stage* [Fig. 4.5 (*e*)] the pipe is filled with concrete.

5. Western button bottom pile.

These piles are used in locations where increase in the end bearing area is desired. The pile uses a concrete plug, of the shape of a button. This button forms and enlarged hole in the soil during driving. Due to this, the side friction is reduced temporarily. These piles have been used upto lengths of about 23 m, and for loads upto 50 tonnes. The four stages in the pile driving are shown in Fig. 4.6.

In the *First Stage* [Fig. 4.6 (*a*)], a steel pipe, with 12 mm thick walls and reinforced base of cast steel, is set over the concrete

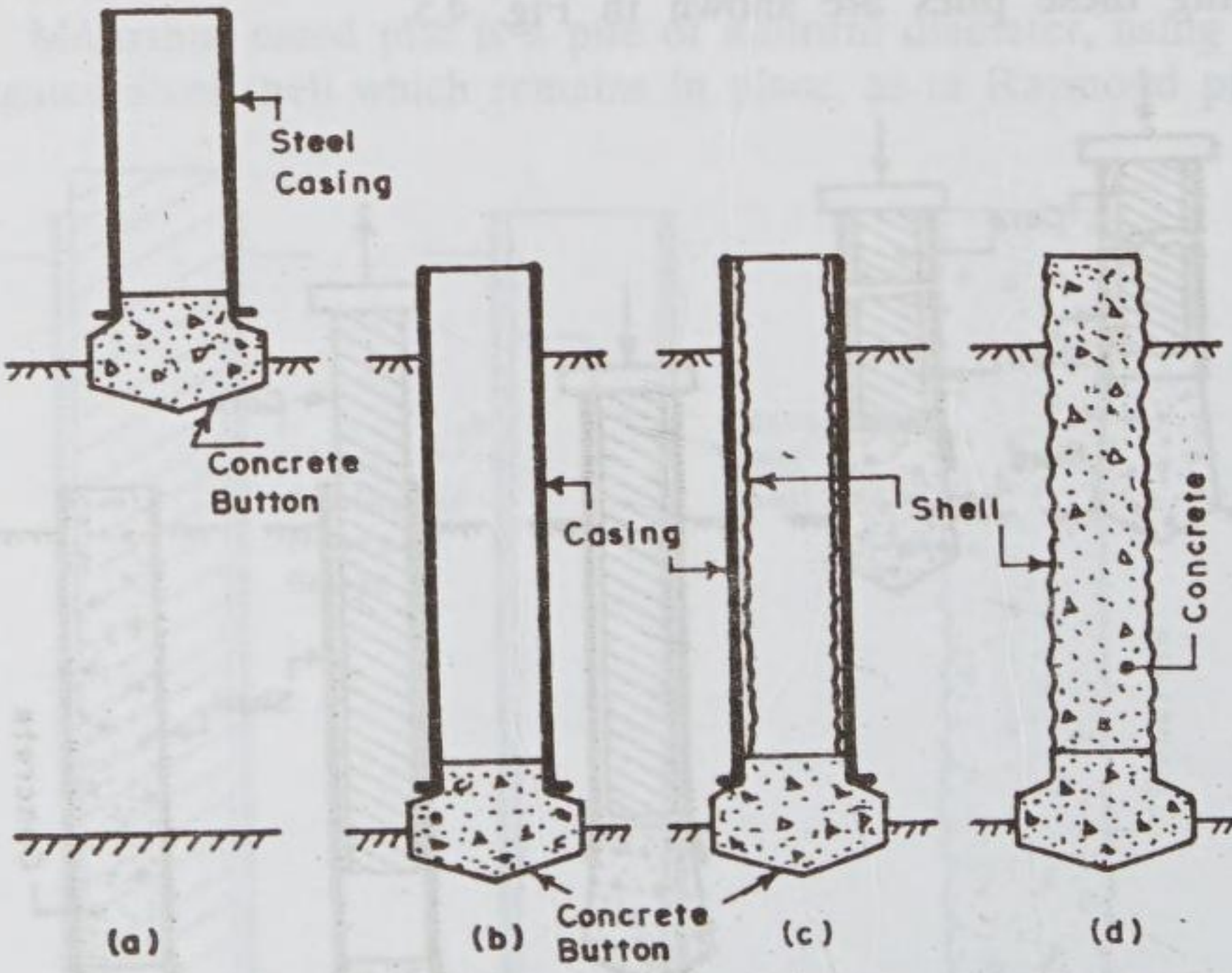


FIG. 4.6. STAGES IN BUTTON-BOTTOM PILE CONSTUCTION.

button. The concrete button has a diameter about 25 mm larger than the pipe. In the *second stage* [Fig. 4.6 (*b*)] the pipe and button are driven to a specified depth. In the *third stage* [Fig. 4.6 (*c*)] a

corrugated steel shell is inserted in the pipe, resting on the button. A steel plate with a bolt hole in it is welded on the bottom of the shell, before lowering it, so that the hole may fit over the central bolt in button bottom. The nut may be tightened with the help of a long socket wrench. In the *fourth stage* [Fig. 4.6 (*d*)], the casing is withdrawn, leaving the button in place, and the shell is filled with concrete. Reinforcement may be used if necessary.

4.4. UNCASED CAST-IN-SITU CONCRETE PILES

These piles do not use casing, and hence are cheaper. However, great skill is required in their construction. These piles are used only where it is certain that neither soil nor water will fall into the hole, or squeeze into and reduce the size of the hole left after withdrawing a driven mandrel or shell before concreting, and also where adjacent piles will not damage the green concrete. It is essential to have close installation inspection, since no inspection is possible after they are installed. These piles have the advantage that (*i*) they need no storage space, (*ii*) they do not require cutting off excess lengths or building up short lengths, (*iii*) they do not require special handling equipment, and (*iv*) the concrete is not liable to damage from driving. The following are the common types of uncased cast-in-situ concrete piles :

(*i*) Simplex pile
(*ii*) Franki pile
(*iii*) Vibro piles and
(*iv*) Padestal piles.

1. Simplex pile.

Simplex pile can be driven through soft or hard soils. In this pile, a steel tube fitted with a cast iron shoe is driven into the ground up to the required depth, as shown in Fig. 4.7(*a*). Reinforcement, if necessary, is put inside the tube. Concrete is then poured into the tube, and the tube is slowly withdrawn, without concrete being tamped, leaving behind the cast iron shoe. Fig. 4.7(*c*) shows the completed pile. The soil must be sufficiently firm to form a good mold for green concrete after the casing is withdrawn, or else an inner casing of slightly smaller diameter than the shell must be inserted before pouring the concrete. This pile is known as *Simplex standard pile* . If, however, tamping of concrete is done at regular interval as the tube is withdrawn, we get the *Simplex tamped pile*

In the above method, the cast iron shoe remains behind, and a new shoe has to be used for each pile. But sometimes, if the soil is firm enough to stand, the cast iron shoe is provided with *alligator jaw point* [Fig. 4.8 (*a*)]. This jaw point is hinged to the shell. When the concrete is poured, the jaw opens and allows concrete to flow out down into the hole. The jaw point is withdrawn gradually

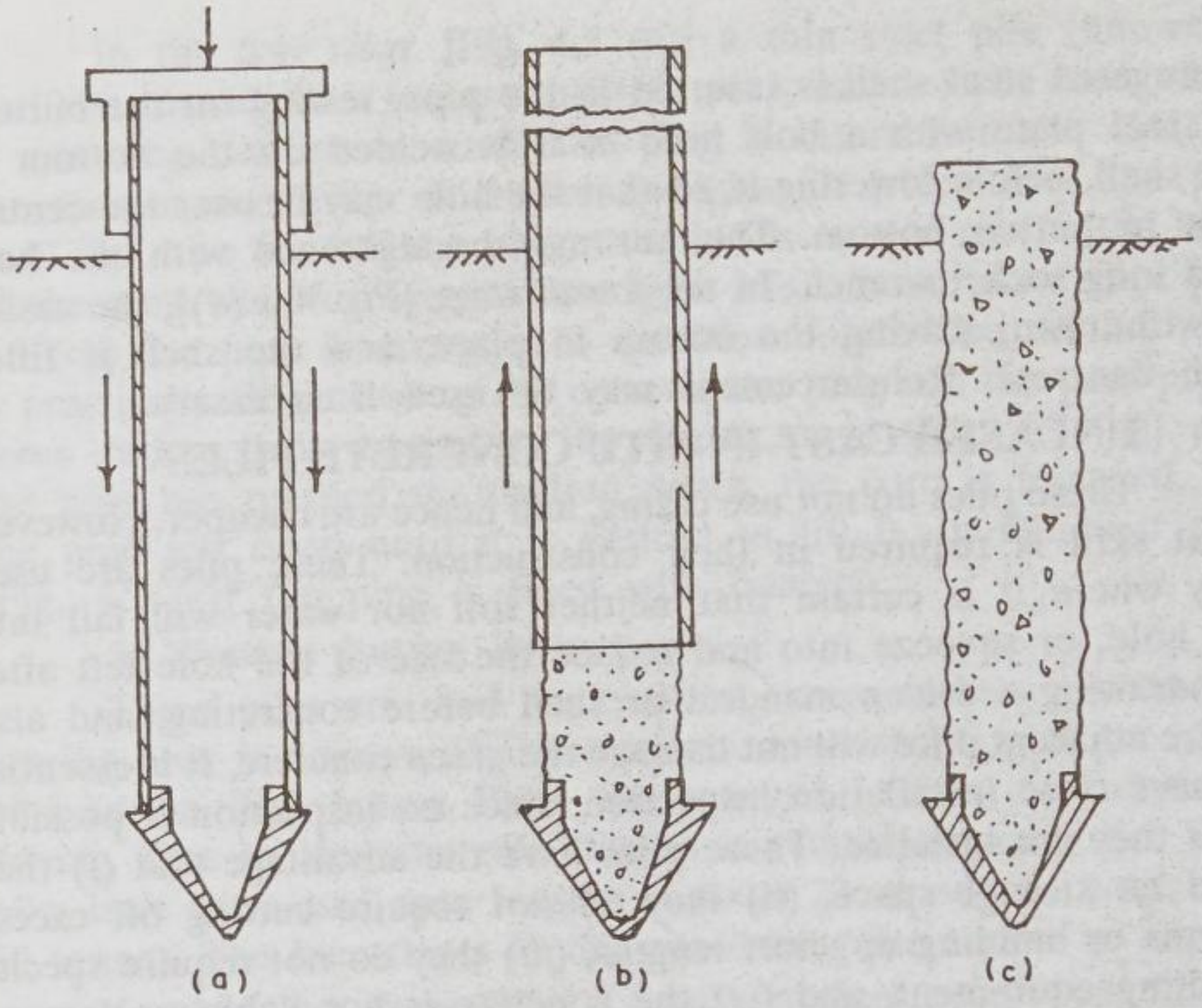


FIG. 4.7. SIMPLEX STANDARD PILE

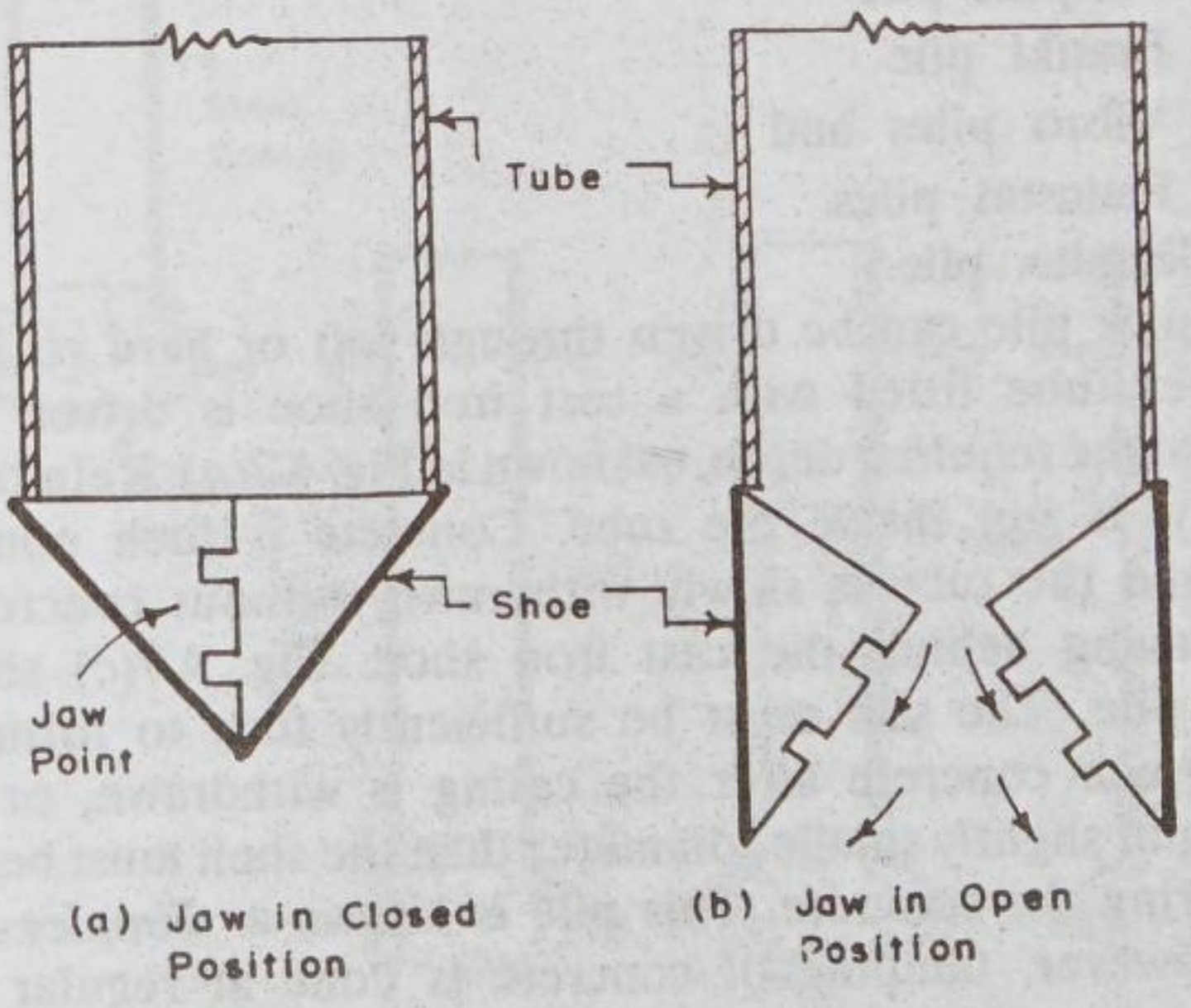


FIG. 4.8. ALLIGATOR JAW POINT.

along with the steel tube as concrete is filled in. However, it would seem difficult to be certain that no soil would fall into the hole at the bottom.

2. Franki pile.

This pile has an enlarged base of mush-room shape, which gives the effect of a spread footing. This pile is more useful where a bearing stratum of limited thickness can be reached at reasonable depth. Also, this type of pile is best suited to granular soil. Fig. 4.9 shows various stages of forming the pile.

In the *first stage,* a heavy removable pipe shell is set vertically on the ground with the help of leads [Fig. 4.9 (*a*)] and a charge of dry concrete or gravel is formed. In the second stage, a diesel operated drop hammer of 20 to 30 kN weight is driven on the concrete. This results in the formation of a dense plug that penetrates the ground and drags the tube with it on account of friction developed between the tube and the concrete plug [Fig. 4.9(*b*)]. In the third

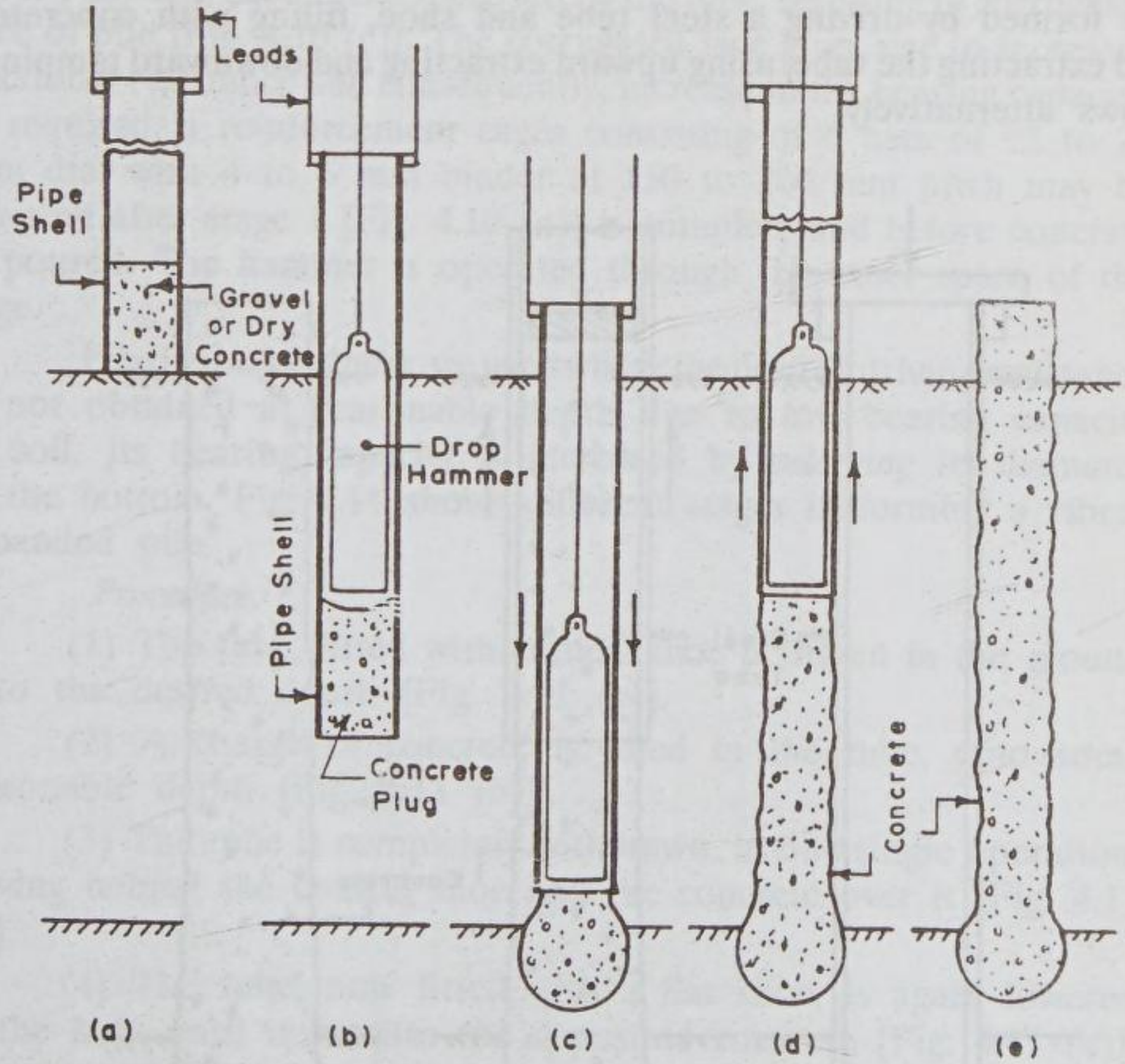


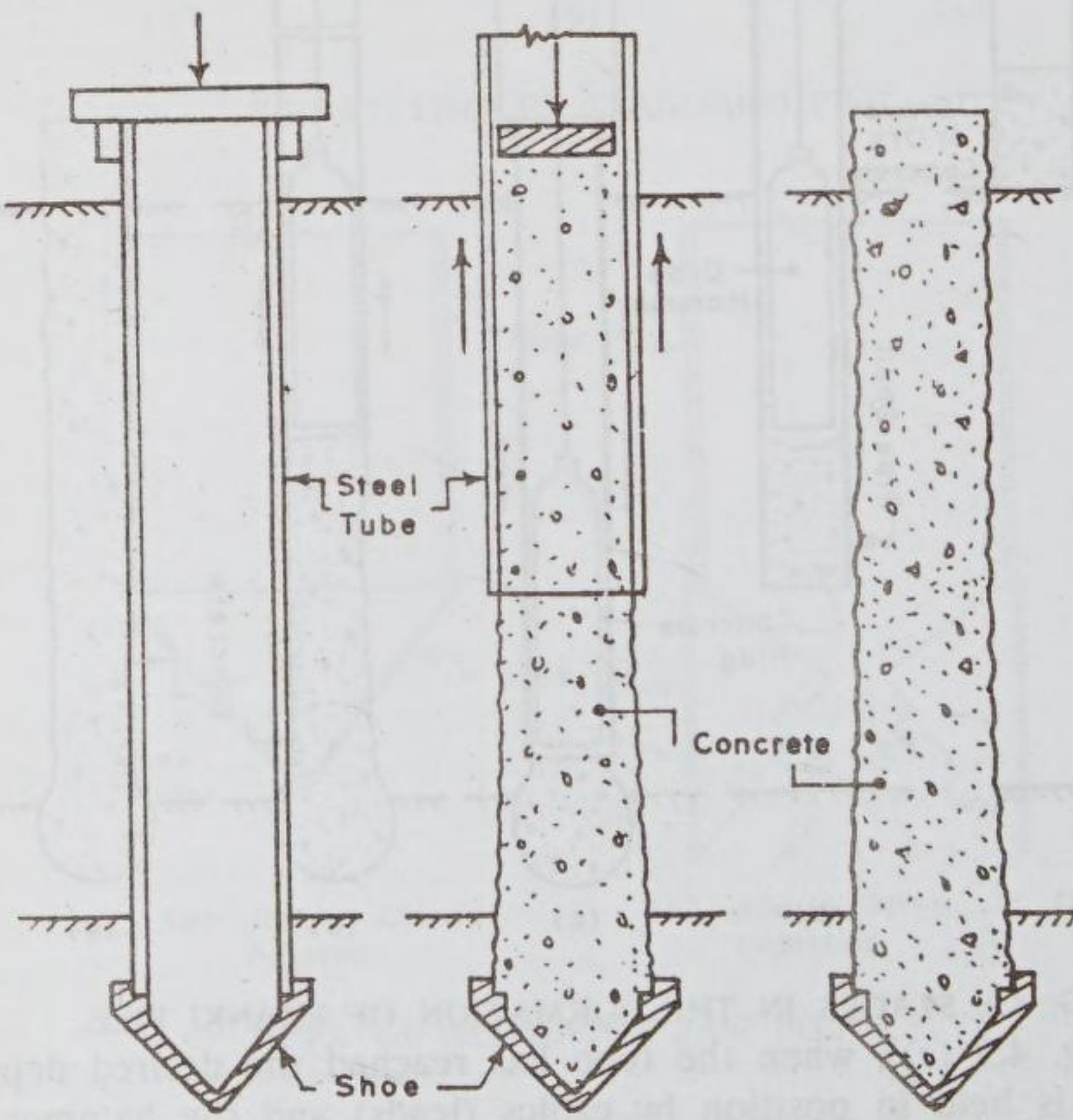
FIG. 4.9. STAGES IN THE FORMATION OF FRANKI PILE.

stage [Fig. 4.9 (*c*)], when the tube has reached the desired depth, the tube is held in position by cables (leads) and the hammer is applied to the concrete, forcing it down and outward. This results in the enlargement of the base into the mushroom shape. If required, a fresh charge of semi-dry concrete is put to enlarge the bulb. In

the fourth stage, the shaft is formed by introducing successive charges of concrete, ramming each in turn, and withdrawing gradually (about 300 mm at a time) the casing [Fig. 4.9 (*d*)]. Fig. 4.9 (*e*) shows the finally formed pile which has corrugations all along its height. Reinforcement cage can be installed, if desired, after the enlarged base has been formed (Stage 3). In that case, the hammer goes inside the cage of reinforcement. The pile diameters in Franki piles vary from 50 cm to 60 cm, while the enlarged base may have a diameter of about 90 cm or more. The pile has a carrying capacity of 60 t (600 kN) to 90 t (900 kN).

3. Vibro-piles.

These piles are used where the ground is soft, thus offering little frictional resistance to the flow of concrete. Both '*standard*' and '*expanded*' piles are formed by the vibro-process. Vibro piles are formed by driving a steel tube and shoe, filling with concrete, and extracting the tube, using upward extracting and downward tamping blows alternatively.



.FIG. 4.10. STAGES IN THE FORMATION OF STANDARD VIBRO-PILES

The three stages of formation of *standard vibro-piles* are shown in Fig. 4.10. *standard vibro-piles* are made in size of 35, 45 and 50 cm dia., the larger for loads of 60 to 70 t (600 to 700 kN) respectively. They can be formed in the lengths of 25 m and over. A steel tube, fitted with (but not fixed to) a cast iron shoe is driven in the ground by 2 to 2.5 t (20 to 25 kN) hammer, operated by steam or compressed air delivering upto 40 blows per minute with a stroke of upto 1.4 m. When the shoe and the tube has reached the desired level [Fig. 4.10 (*a*)], corresponding to the desired set, extracting links are fitted to the hammer and top of the tube. The tube is now filled with concrete (usually 1 : 2 : 4 mix). The withdrawal of the tube and the ramming of the concrete are effected by hammer operating at 80 blows per minute. Each up-stroke results in 4 cm withdrawal of the tube (leaving the shoe behind) while concrete is consolidated in each downward blow. Thus the concrete is being forced down to occupy the space left by the tube, resulting in corrugated face of pile [Fig. 4.10 (*c*)]. The corrugated face gives rise to increased frictional resistance and consequently, increase in the bearing capacity. If required, a reinforcement cages consisting of 6 bars of 12 to 24 mm dia. with 4 to 6 mm binder at 150 to 200 mm pitch may be lowered after stage 1 [Fig. 4.10 (*a*)] is complete and before concrete is poured. The hammer is operated through the inner space of the cage.

Vibro-expanded piles are used where the desired driving resistance is not obtained at reasonable depth due to low bearing capacity of soil. Its bearing capacity is increased by *enlarging* its diameter at the bottom. Fig. 4.11 shows different stages in forming a vibro-expanded pile.

Procedure.

(1) The tube, fitted with conical shoe is driven in the ground upto the desired depth [Fig. 4.11 (*a*)].

(2) A charge of concrete is filled in the tube, upto some reasonable depth [Fig. 4.11 (*b*)].

(3) The tube is completely withdrawn, in one single operation, leaving behind the conical shoe and the concrete over it [Fig. 4.11 (*c*)].

(4) The tube, now fitted with a flat shoe, is again lowered in the hole until it reaches the deposited concrete [Fig. 4.11 (*d*)].

(5) The tube is driven down along with flat shoe, to penetrate into the concrete, resulting in the formation of bulged end. [Fig. 4.11(*e*).

(6) The reinforcement cage is lowered in the tube, so that it rests on the flat shoe, (which is positioned at about a metre or two above the conical shoe), as shown in Fig. 4.11 (*f*).

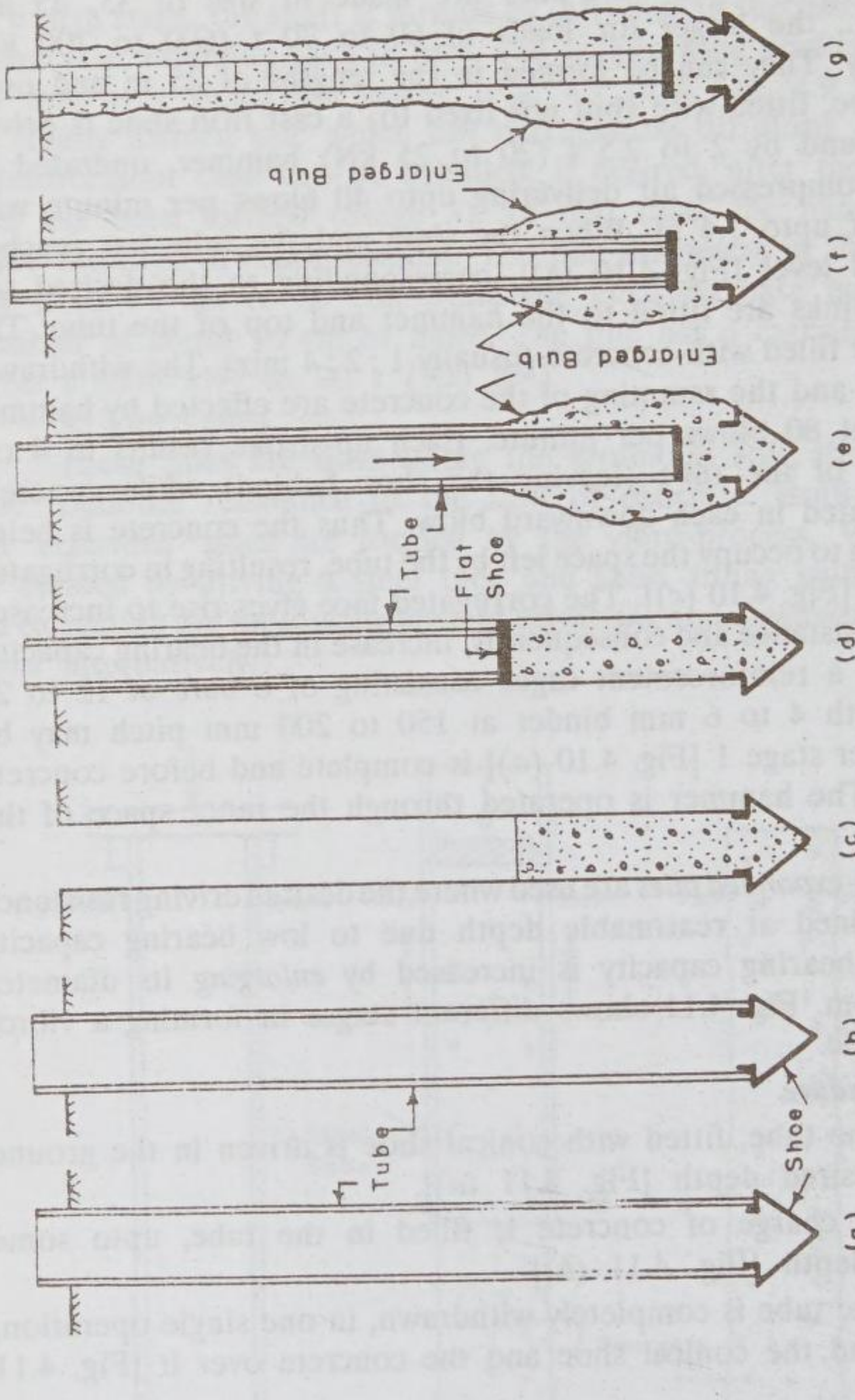


FIG. 4.11. STAGES IN FORMING VIBRO-EXPANDED PILE.

(7) The tube is now fitted with concrete, and the pile is completed by succession of upward extracting and downward consolidating blows, as in the standard vibro pile. Fig. 4.11(*g*) shows the finally formed pile.

4. McArthur padestal piles.

The pile is used where thin bearing stratum is reached with reasonable depth. The padestal of the pile gives the effect of spread footing on this comparatively thin bearing. The pile uses a steel tube casing and a steel core, the lower end of the core being flush

with the bottom of the casing and the end made flat. The stages of forming the padestal pile are shown in Fig. 4.12.

Procedure.

1. The core and casing are driven together into the ground, till they reach the required level [Fig. 4.12 (*a*)].

2. The core is taken out, and a charge of concrete is placed in the tube [Fig. 4.12 (*b*)].

3. The core is replaced in the casing till it rests on the top of the poured concrete. While maintaining a pressure of the core and the hammer on the concrete, the casing is pulled up by 50 cm to 1 m [Fig. 4.12 (*c*)].

4. The charge of concrete is rammed out, thus resulting the formation of a padestal [Fig. 4.12 (*d*)].

5. The core is removed, the casing is filled with concrete, and core is replaced in contact with concrete [Fig. 4.12 (*e*)].

6. The casing is pulled up while maintaining the pressure of core and hammer on the concrete. The finished pile is shown in Fig. 4.12 (*f*).

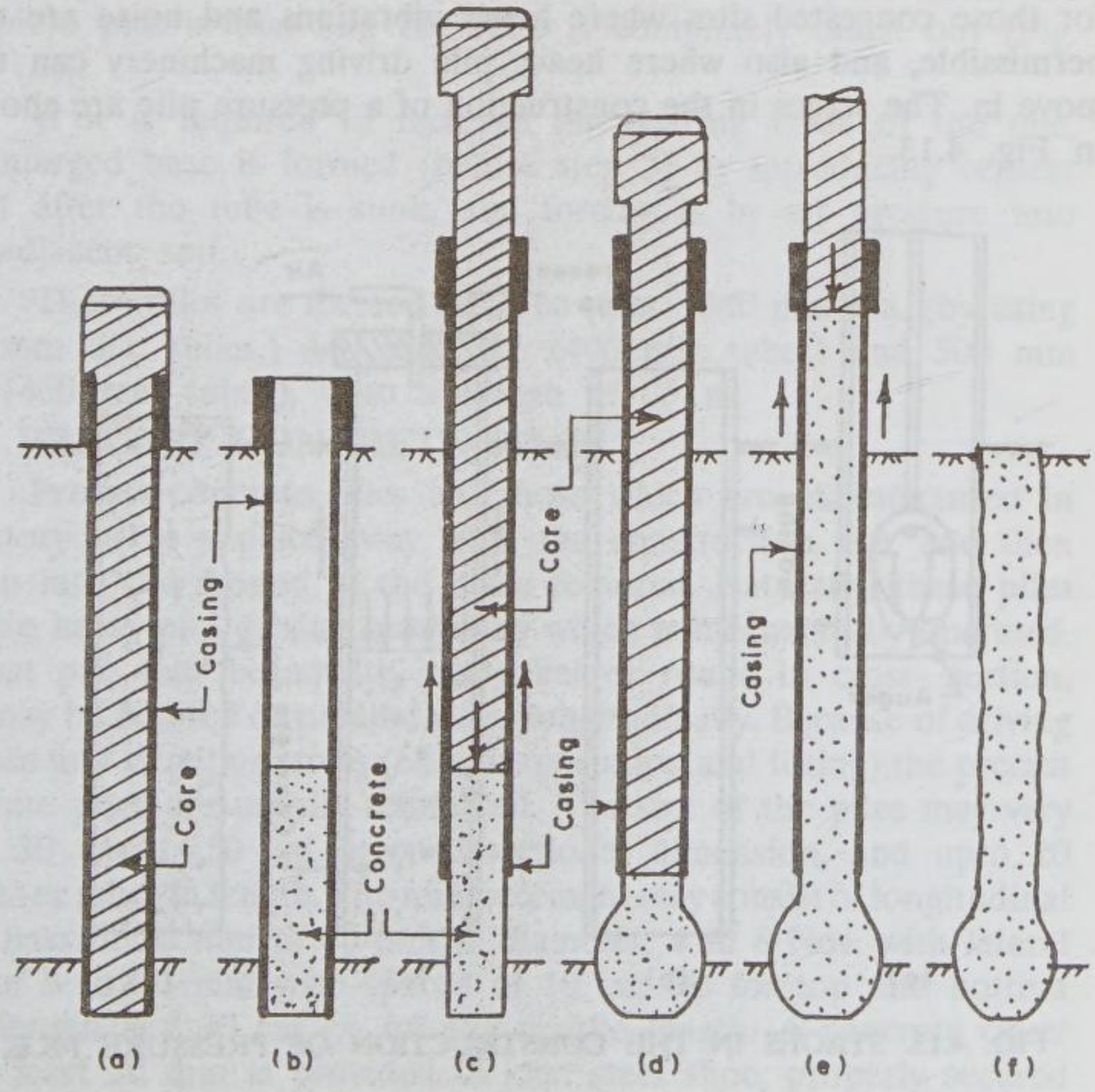


FIG. 4.12. STAGES IN THE FORMATION OF Mc-ARTHUR PADESTAL PILES.

4.5 BORED PILES

Bored piles are those which are formed by forming a bore hole in the ground and then concreting it, either with the help of a casing tube or without a casing tube. Their procedure of construction is thus different than the cast-in-situ driven pile where a heavy pile driving equipment is required. Evidently, these piles have advantage over the driven piles, in those locations and those situations where the vibrations and noise caused by driving of piles are to be avoided or the strata of adequate bearing capacity is so deep that they are difficult to reach by *driven piles.*

Bored piles are of three types :

1. Pressure piles
2. Under-reamed piles and
3. Bored compaction piles.

Under-reamed piles and bored compaction piles have been discussed in § 4.16 and § 4.17 respectively.

PRESSURE PILES

They are formed with the help of a casing tube, boring auger and compressed air equipment. These piles are especially suitable for those congested sites where heavy vibrations and noise are not permissible, and also where heavy pile driving machinery can not move in. The stages in the construction of a pressure pile are shown in Fig. 4.13.

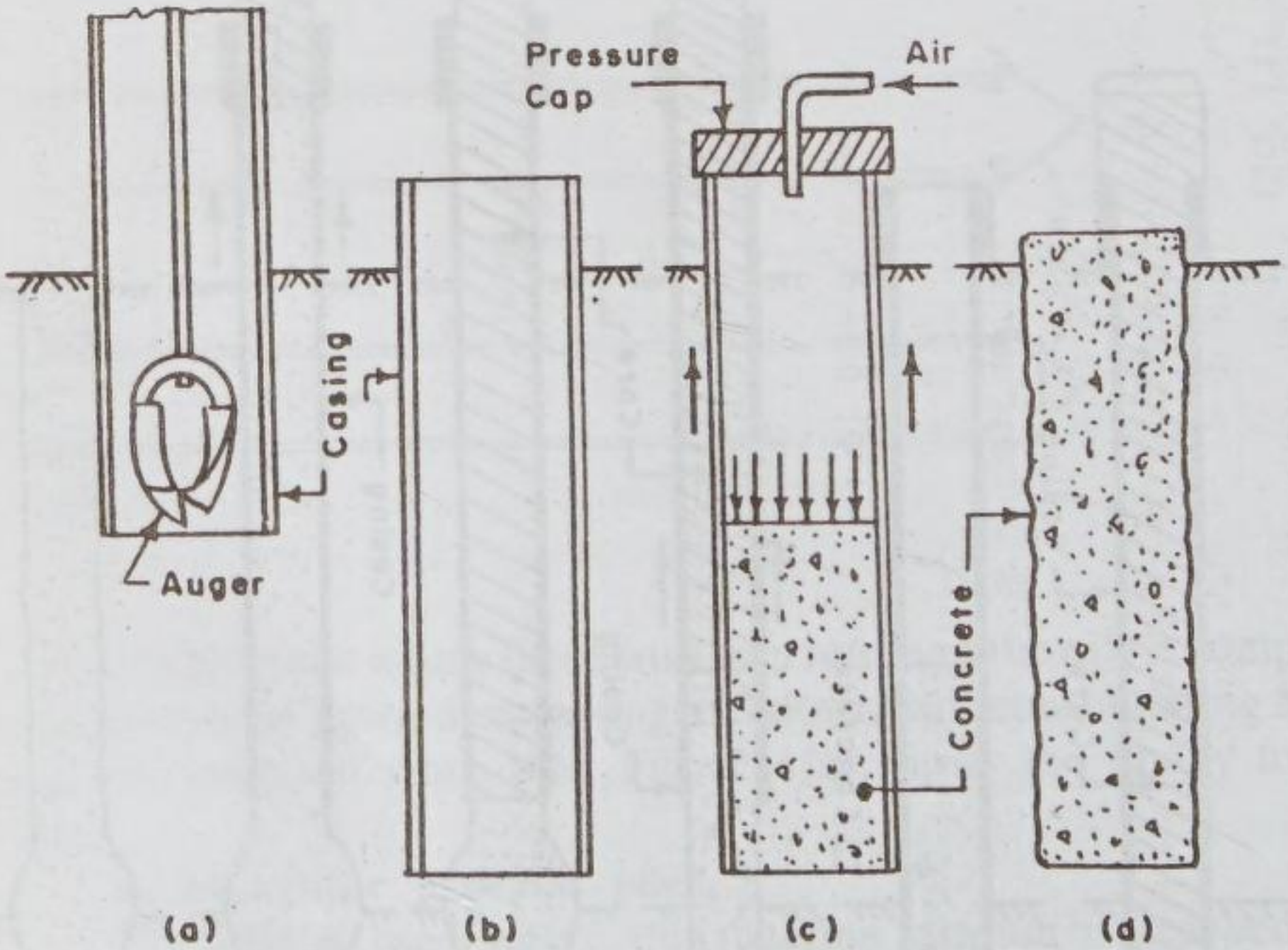


FIG. 4.13. STAGES IN THE CONSTRUCTION OF PRESSURE PILE.

Procedure.

1. A 1.2 to 1.8 m long section of steel tube, 400 mm is dia, is sunk in the ground, whilst a boring tool, such as an auger, working inside it, excavates the soil [Fig. 4.13 (*a*)].

2. Further sections of 1.2 to 1.8 m long steel tubes are screwed successively and sunk in the ground, as boring proceeds, till the required depth is reached. The bored soil is continuously taken out. At the end, the boring tool is taken out and the hole is cleaned [Fig. 4.13 (*b*)].

3. A charge of concrete is placed in the tube, and the upper end of the tube is closed with the help of pressure cap. Compressed air is introduced through the air pipe of the pressure cap, thus forcing the concrete down and out against the surrounding soil. Simultaneously, the tube is slowly extracted with the help of a winch [Fig. 4.13 (*c*)]. The diameter of the pile exceeds that of the tube owing to the compression of soil, and rough irregular surface is formed which increases the frictional resistance of piles .

4. Fresh charges of concrete are placed in the tube, before the end of the tube comes above the previous charge of concrete, and the process of compressed air application is repeated, till the complete pile is cast and the tube is completely taken out [Fig. 4.13 (*d*)].

If it is required to increase the bearing value of the pile, an enlarged base is formed (before step 3) by introducing cement grout after the tube is sunk, and forcing is by air pressure into the adjacent soil.

These piles are formed in three sizes : 340 mm dia. (by using 300 mm dia. tubes,) 440 mm dia. (400 mm tubes) and 500 mm dia. (460 mm tubes), upto a length of 25 m.

4.6. PRECAST CONCRETE PILES

Precast concrete piles are those which are manufactured in a factory or at a place away from the construction site, and then driven into the ground at the place required. Naturally, these piles require heavy pile driving machinery which is mechanically operated. Precast pile may be square, octagonal or round in cross- section, and may be tapered or parallel sided longitudinally. Because of driving stresses and handling stress (*i.e.* transportation and lifting),the precast concrete piles are usually reinforced. The size of the piles may vary from 30 cm to 50 cm in cross-sectional dimension, and upto 20 metres or more in length. The reinforcement may consist of longitudinal steel bars of 20 mm to 40 mm in diameter, 4 to 8 Nos. with lateral ties of 5 to 10 mm wire spaced at 10 cm c/c for top and bottom 1 m length and 30 cm c/c for the middle length. A concrete cover of at least 50 mm is provided. A cast steel shoe, properly secured

to the pile by mild steel straps, is provided at its lower end. The shoe protects the pile toe and helps the pile in penetrating into hard soil during driving. Fig. 4.14 shows the typical cross-sections of square, octagonal and circular pre-cast piles of concrete. However, square and octagonal sections are most common because these shapes are easy to cast in horizontal position.

Precast concrete piles are useful in carrying fairly heavy loads through soft material to firmer strata. They require time for curing, storage space and equipment for handling. It may be difficult to predetermine lengths, which may involve large expenditure in cutting off (excess length) or building up (short length). The cross-section and reinforcement are usually governed by handling stresses. These *stressses* depend upon the method of lifting and the location of points of support. For piles upto 8 m length, one point of support is sufficient. For longer piles, more points (*i.e.* 2 or 3 or even more) are used to reduce the handling stresses thus resulting in more economical section. When pile is *suspended* from *one point*, that point is located at its mid-length. When pile is *suspended* from *two points*, each point is located at a distance equal to 0.206 L from either end, where L is the length of the pile. When pile is *suspended* from *three-points* each end point is located at a distance of 0.15 L from the respective ends while the third point is provided at its mid-length. However, when pile is *erected* from one point, that point should be located at a *distance* of 0.293 L from the head of the pile. These supporting points and lifting points should be clearly pointed on the pile.

Procedure for forming precast concrete piles:

1. The form work of the required shape is prepared. Usually, metal forms are used for mass manufacture. The inner sides of the form is coated with either soap solution or oil so that concrete does not adhere to the sides.

2. The reinforcement cage, as per design, is placed in the form, maintaining proper cover all around. Cast steel shoe is also placed, and is secured to the reinforcement with the help of mild steel straps.

3. Concrete is then placed in the form and well vibrated with the help of form vibrators. The usual mix of concrete is 1 : 2 : 4, with maximum size of aggregate equal to 19 mm. However, if high driving conditions are to be encountered, rich mix ($1 : 1\frac{1}{2} : 3$) may be used. When the pile is driven it is subjected to greatest impact stresses at its head, and it is sometimes strengthened at the upper 0.6 to 0.9 m length by the use of stronger grade of concrete (1 : 1 : 2)

4. Remove the form only after 3 days. However, the piles are kept in the same position atleast for 7 days. The piles are then

shifted to the curing tank where concrete is allowed to mature for atleast 4 weeks before being driven. This period can be reduced to one week if, instead of normal Portland cement, rapid hardening cement is used.

Advantages of precast concrete piles

1. The piles are manufactured in the factory. Hence proper control can be exercised over the composition and design of these piles. High grade concrete can be used because of controlled conditions.

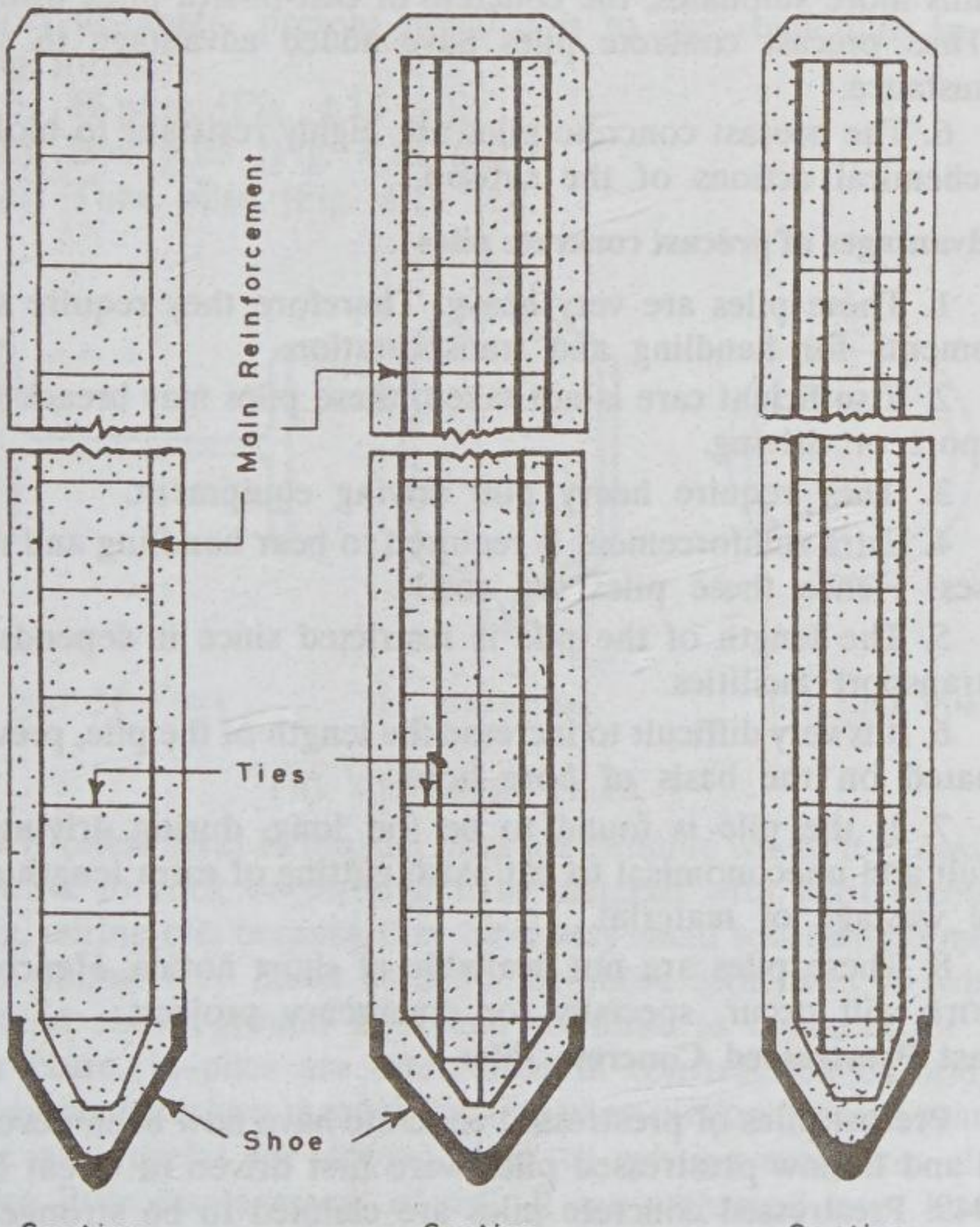


FIG. 4.14. PRECAST CONCRETE PILES

2. The position of reinforcement in the pile is not liable to be disturbed.

3. The casting defects can be easily discovered after the removal of forms, and these defects (such as hollows, etc.) can be properly repaired before driving the pile.

4. Since a large number of piles are manufactured at a time, in the factory or any other convenient place,the cost of manufacturing these will be less.

5. These piles can be driven under water. If the subsoil water contains more sulphates, the concrete of cast-in-situ piles would not set. Thus precast concrete piles have added advantage in such a circumstance.

6. The precast concrete piles are highly resistant to biological and chemical actions of the subsoil.

Disadvantages of precast concrete piles

1. These piles are very heavy. Therefore they require special equipments for handling and transportation.

2. If sufficient care is not taken, these piles may breack during transport or driving.

3. They require heavy pile driving equipment.

4. Extra reinforcement is required to bear handling and driving stresses. Hence these piles are costly.

5. The length of the pile is restricted since it depends upon the transport facilities.

6. It is very difficult to increase the length of the pile, previously estimated on the basis of bore holes.

7. If the pile is found to be too long, during driving, it is difficult and uneconomical to cut. Also cutting of extra length results in the wastage of material.

8. These piles are not available at short notice. Hence delay of work will occur, specially for emergency projects.

Precast Prestressed Concrete Piles

Precast piles of prestressed concrete have now been developed. Solid and hollow prestressed piles were first driven in Great Britain in 1949. Prestressed concrete piles are claimed to be stronger than the normal reinforced concrete piles and, therefore, because of the reduction in the cross-sectional area, they are lighter and can be more easily handled. A prestressed concrete pile has the following advantages: (*i*) it has greater ability to withstand extremely hard driving (*ii*) it is more durable in sea water because of absence of cracks (*iii*) it has greater column capacity (*iv*) it has lesser handling costs because of light weight (*v*) it requires lesser pick up points,

and (*vi*) it has much larger moment of inertia then the conventional piles of the same dimensions since the concrete is all in compression.

4.7. STEEL PILES

A steel pile may be a rolled section, a fabricated shape or a piece of sheet pile. Two or more sections of sheet piles may be connected together in a box shape and driven as one pile. Metal piles have been used since 1838, in the form of cast iron pipes or solid wrought iron shafts with disks or screw flanges penetrating only short distances. present practice is to use steel piles in the following forms:

(*i*) H-piles [Fig. 4.15 (*a*)]
(*ii*) Box piles [Fig. 4.15 (*b*)].
(*iii*) Tube piles [Fig. 4.15 (*c*)].

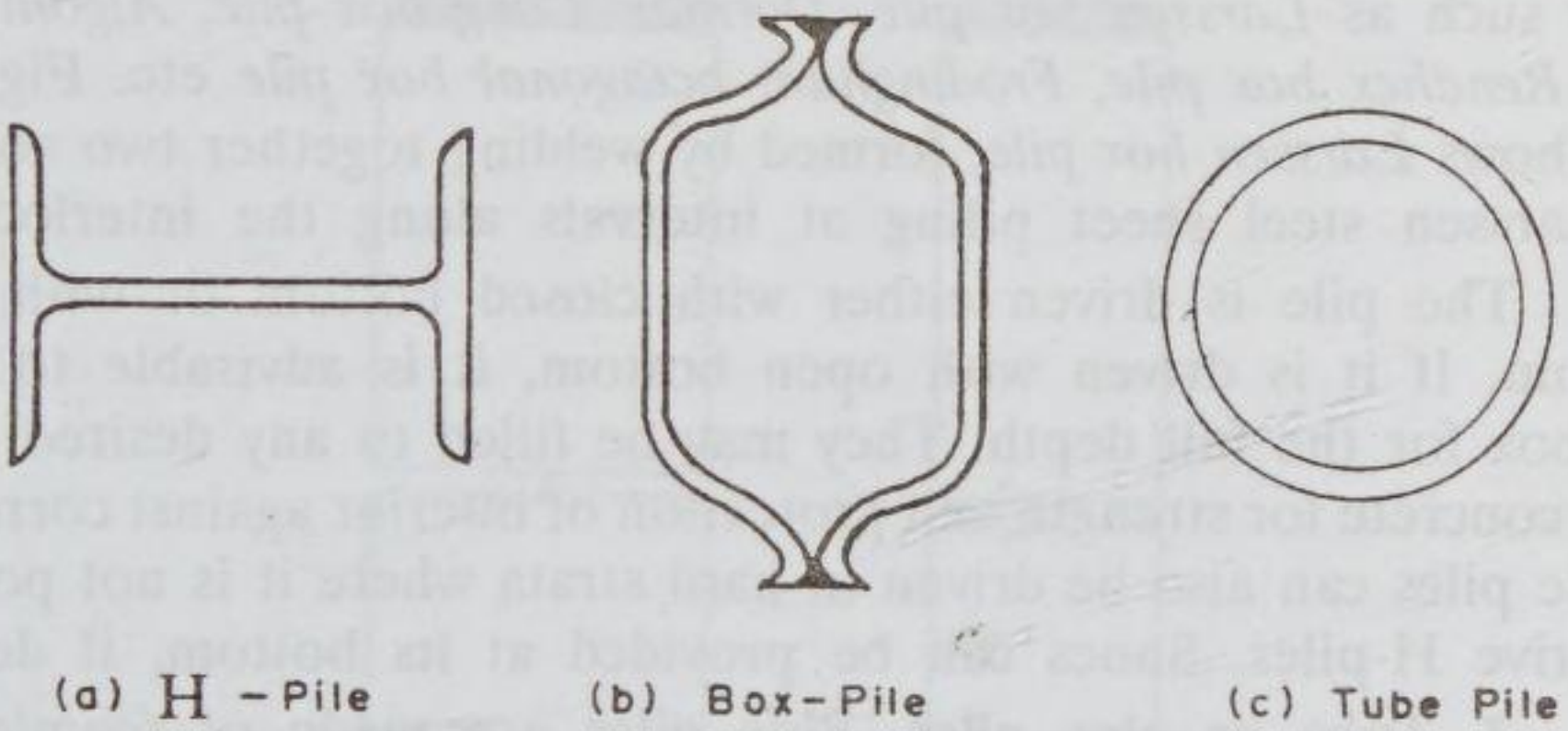


FIG. 4.15. STEEL PILES.

1. H-piles. Steel H bearing piles are suitable where it is desired to penetrate to rock or through hard material with least recourse to jetting, coring etc. because they have very small soil displacement. They are found very much suitable for those structures in which piles extend above ground level and continue as columns for the super-structure. H–piles are often used in construction of bridges where they can be driven through existing construction in small spaces. They are also useful for driving close to existing structures since they cause little displacement of soil. It can withstand large lateral forces. They require less space for shipping and storing than wood, pipe or precast concrete piles. Also, they do not require special slings or special care in handling.

H-piles can be spliced in the same manner as steel columns. The splices can be riveted, bolted, or welded. It is customary to design the splice to resist a moment equal to one-third to one-half of the moment capacity of the H–section. The flanges and the web of H-piles are rolled with equal thickness in order to eliminate damage on thinner part. The flange width is made atleast 85 percent

of the depth of the pile section in order to provide rigidity in the weak axis. Steel plates are welded on the top of H-piles to transfer the pile loads to the concrete pile cap. The driving point (end of H-pile) may be reinforced and strengthened by adding welded or riveted plates. Cast steel points are sometimes valuable for piles which are to be pulled and redriven several times but are of no advantage for permanent piles. It is not recommended that points be chamfered or sharpened, unless driving to bearing on sloping surfaces of rock. However, blunt flat ends drive straighter and penetrate faster into soft rock and hard soils.

2. Box piles. Box piles, formed of steel sheeting with or without deep beams have great lateral strength. Such piles are generally used to support a wharf or other sea structures where deep water, silt and sliding banks are present. There are various forms of box piles such as *Larssen box pile*, *Dorman Long box pile*, *Algoma box pile, Rendhex box pile*, *Frodingham octagonal box pile* etc. Fig. 4.15 (*b*) shows *Larssen box pile*, formed by welding together two sections of Larssen steel sheet piling at intervals along the interlocks.

The pile is driven either with closed bottom or with open bottom. If it is driven with open bottom, it is advisable to clean the box for the full depth. They may be filled to any desired depth with concrete for strength and protection of interior against corrosion. These piles can also be driven in hard strata where it is not possible to drive H-piles. Shoes can be provided at its bottom, if desired.

3. Tube or pipe piles. Pipe piles are made of seamless or welded pipes, which may be driven either closed-ended or open ended. When driven with open end, the material inside the pipe is removed by suitable method, and concrete is then filled inside for strength and protection of interior against corrosion. The closed-end piles are formed by fixing a driving point to the tip of the pile. The choice between open end and closed end types depends upon the soil conditions at the site.

4.8. TIMBER PILES

A timber pile is made of the trunk of a tree, trimmed of branches. It must satisfy the following requirements: (*i*) freedom from sharp bends, large or loose knots, shakes, splits and decay (*ii*) freedom from short or reverse bends and from crooks greater than one-half the diameter of the pile at the middle of the bend (*iii*) straight line between corners of butt and tip within the body of the pile and (*iv*) uniform taper from butt to tip. The common Indian timbers used for piles are : babul, chir, deodar, jarul, poon, sal, semul, teak white siris and khair. It has been found that piles made from khair wood can stand action of sea water much better and is commonly used for marine works. Untreated piles entirely embedded below

ground water table are considered permanent, provided that marine borers are not present. However, building codes usually prohibit the use of untreated timber piles above water table to support permanent structures. Creosote oil is universally used as preservative.

Timber piles are generally square or circular in cross-section. Fig. 4.16 shows typical timber piles, having cast iron shoe at its bottom. The diameter of circular pile may vary between 30 to 50 cm. Similarly, the size of square pile may also vary between 30 to 50 cm. The length of the pile should not be more than 20 times its top width. These piles are driven with a light pile driving equipment. In order to protect the head of the pile from brooning, an iron ring is fixed at its top. The diameter of the ring is kept about

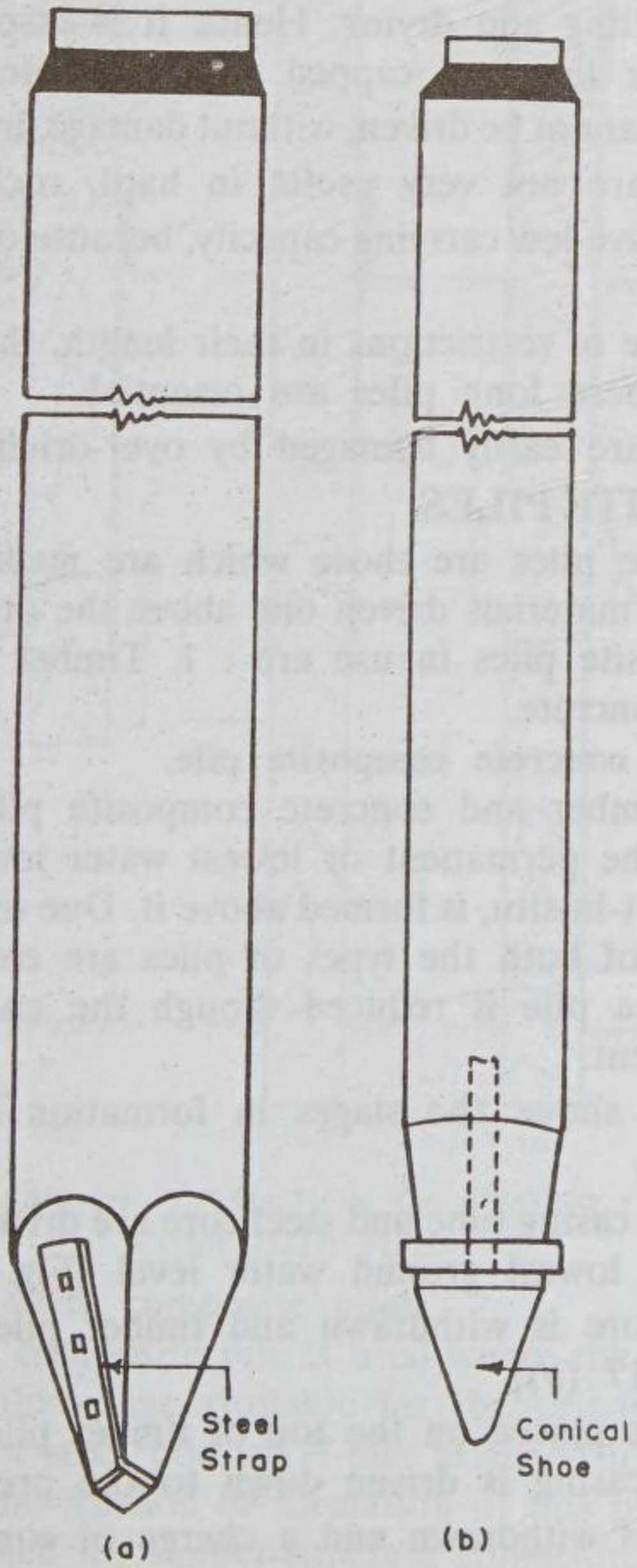


FIG. 4.16. TIMBER PILES.

25 mm less than the diameter or size of the pile head. Timber piles can take loads upto 20 tonnes.

Advantages of timber piles.

1. They are cheap and more economical.
2. They can be easily stored and transported, without the aid of any heavy equipment.
3. They can be driven very rapidly.
4. Because of their elasticity they are better suited to the conditions where piles are subjected to unusual lateral forces.
5. They are specially useful when sub-soil water is present.

Disadvantages of timber piles

1. Timber piles deteriorate or decay very fast when subjected to alternate wetting and drying. Hence it is essential to cut them below the water line and capped with concrete.
2. They cannot be driven, without damage, in made up grounds.
3. They are not very useful in hard, rocky strata.
4. They have low carrying capacity, because of its low structural strength.
5. Because of restrictions in their length, they cannot be used in situations where long piles are essential.
6. They are easily damaged by over-driving.

4.9. COMPOSITE PILES

Composite piles are those which are made of two portions of two different materials driven one above the other. Two common types of composite piles in use are : 1. Timber and concrete and 2. Steel and concrete.

1. Timber and concrete composite pile.

In the timber and concrete composite pile, timber portion is used below the permanent or lowest water level, while concrete piles, usually cast-in-situ, is formed above it. Due to this combination, the advantages of both the types of piles are combined. Also, the total cost of the pile is reduced though the entire length of the pile is permanent.

Fig. 4.17 shows the stages in formation of such a pile :

Procedure:

1. A steel casing tube and steel core are driven into the ground, well below the lowest ground water level [Fig. 4.17(*a*)].
2. The core is withdrawn and timber pile is placed in the casing. [Fig. 4.17 (*b*)].
3. Core is placed on the top of timber pile. The timber pile guided by the casing is driven down to the pre-determined level.
4. Core is withdrawn and a charge of concrete is placed in the casing on the top of the timber pile. The core is replaced over

the top of concrete [Fig. 4.17 (*c*)].

5. Pressure is maintained on the concrete with the help of core and hammer, and the tube is slightly withdrawn, resulting in the formation of a concrete padestal around the top of the timber section [Fig. 4.17 (*d*)]. Thus a proper connection is made between timber and concrete.

6. The remaining portion of concrete pile is formed by putting more concrete in the tube, maintaining pressure over it and withdrawing the tube. Fig. 4.17 (*e*) shows the finally formed composite pile.

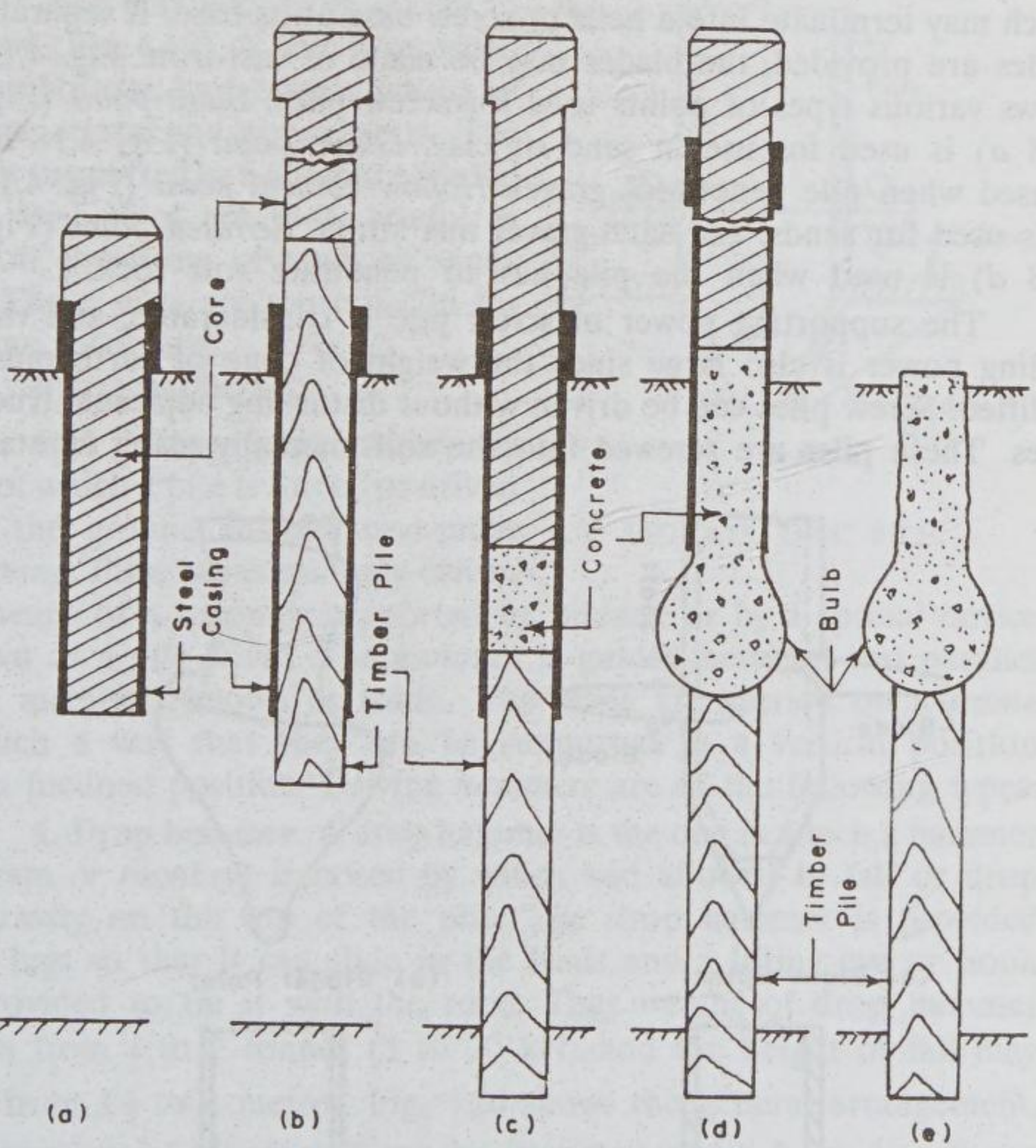


FIG. 4.17. STAGES IN FORMATION OF TIMBER–CONCRETE COMPOSITE PILE.

2. Steel and concrete composite piles.

This type of composite pile is used where the required length of pile is greater than that available for the cast-in-situ type pile. The pile consists of steel pile or H-pile attached to the lower end of concrete pile. The method of formation of this pile is practically the same as that used for timber-concrete composite pile. The steel

H-section is driven first, guided through steel casing tube. The concrete pile is then formed above it, while gradually removing the casing tube, the H-section extends or penetrates at least 1.5 to 2 m into the concrete. Close spacing of surrounding spiral reinforcement is used. This type of composite pile is used where satisfactory penetration of the pile into rock is required for heavy loads.

4.10. SCREW PILES AND DISC PILES

1. Screw Piles.

A screw pile is made of a hollow cast iron or steel shaft. The external diameter of the shaft may vary from 15 to 30 cm, which may terminate into a helix or screw base at its base. If separate blades are provided, the blades may be made of cast-iron. Fig. 4.18 shows various types of points used for screw piles. *Blunt point* (Fig. 4.18 *a*) is used for use in sand or clay. *Gimle point* (Fig. 4.18 *b*) is used when pile penetrates gravel. *Hollow conical point* (Fig. 4.18 *c*) is used for sands, and sand-gravel mix strata. *Serrated point* (Fig. 4.18 *d*) is used when the pile has to penetrate soft rock.

The supporting power of screw pile is considerable, and the pulling power is also large since the weight of cone of earth must be lifted. Screw piles can be driven without disturbing adjacent structures. These piles are screwed into the soil manually using capstan

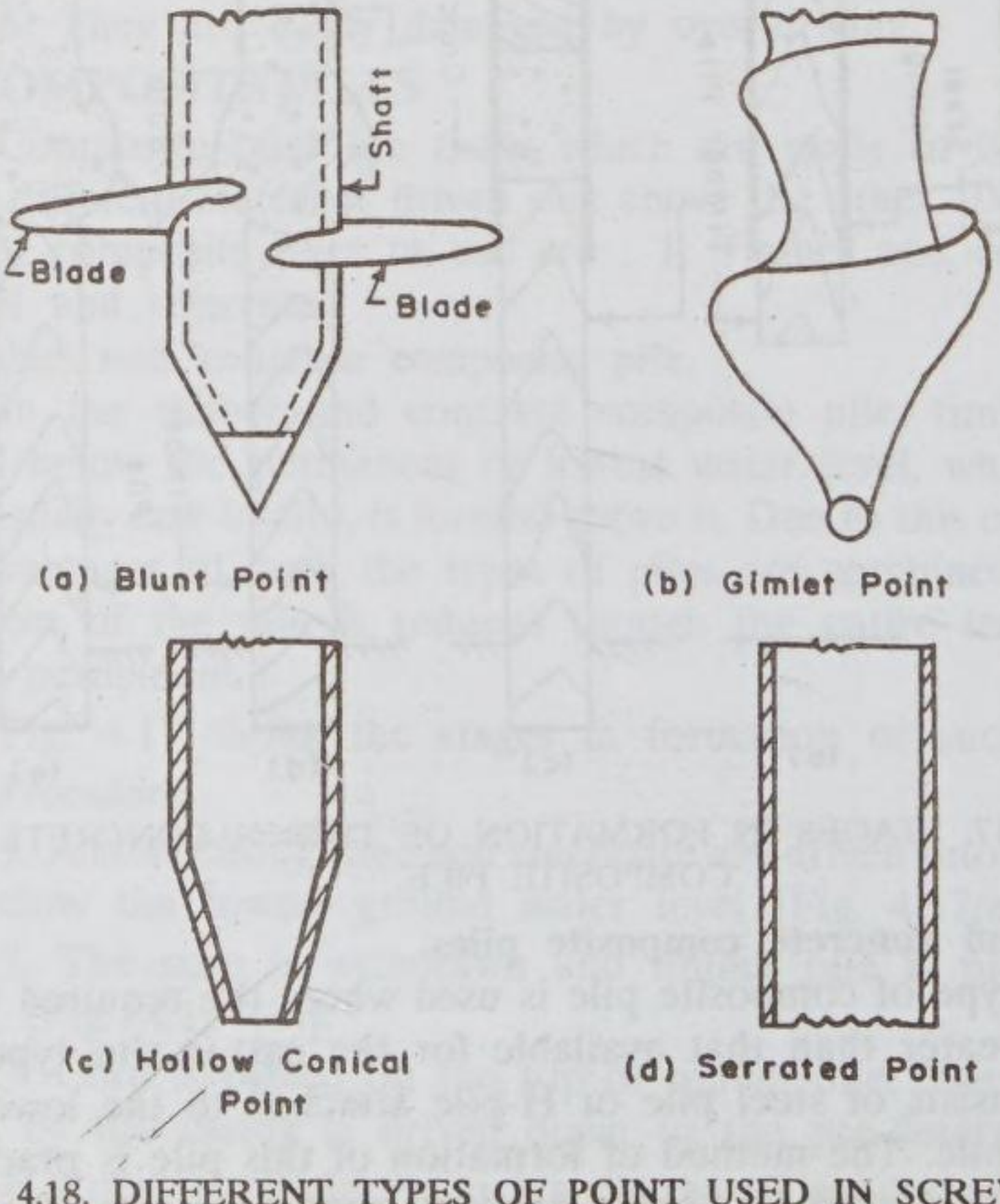


FIG. 4.18. DIFFERENT TYPES OF POINT USED IN SCREW PILES.

bars, or by motive power. The screw penetrates most soils without much difficulty and will push aside boulders that are not too large. In hollow or open–ended points, the soil can be jetted and broken up if screwing becomes too hard. The screw may generally have one or one-and-a-half turn. However, for heavy loads in poor soil, upto three turns of screw have been used.

2. Disc piles.

A disc pile consists of hollow cast-iron pipe with a disc or casting of enlarged size at the bottom, to enlarge the bearing area to a very great extent. The diameter of the disc may vary from 60 cm to 120 cm (Fig. 4.19). A hole is provided at the bottom, to facilitate jetting of harder strata and tough soils. The disc is supported by a number of radial ribs. Disc piles are more useful in subsoil consisting of sands or sandy silt. These piles are more useful for marine structures.

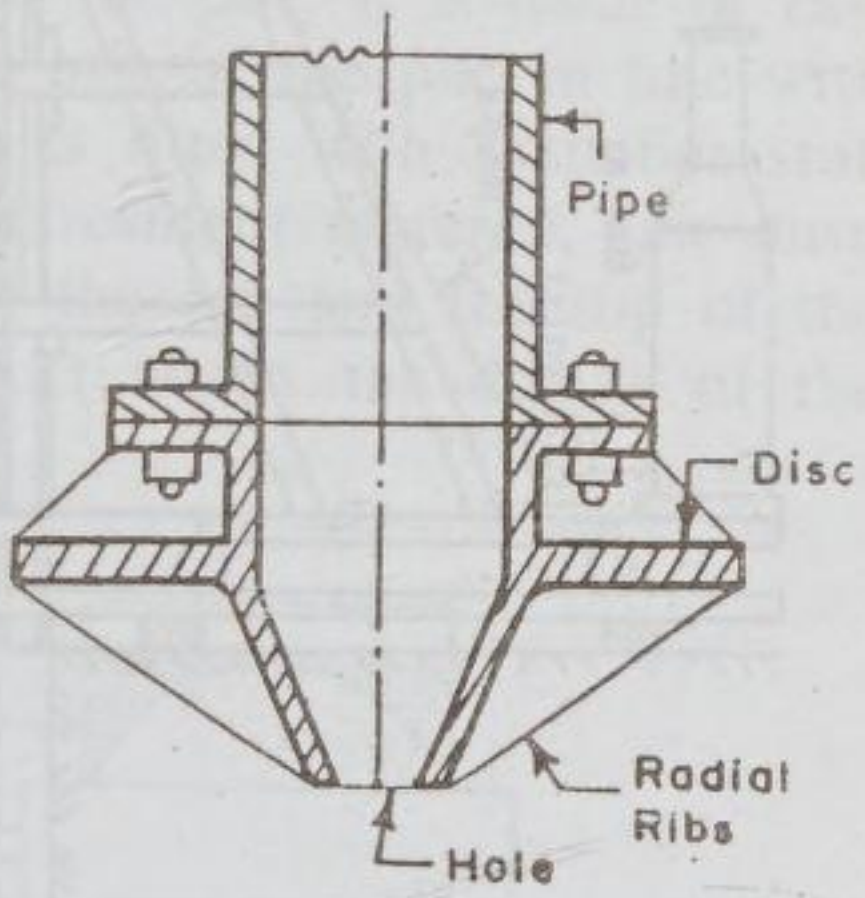


FIG. 4.19. DISC PILE.

4.11. PILE DRIVING

Pile driving is the process by way of which a pile is forced or driven into the ground *without excavation or boring*. Piles are commonly driven by means of a hammer supported by a crane or by a special device known as a *pile driver*. The *hammer* is guided between two parallel steel members known as *leads*.. The leads are carried on a *frame* in such a way that they can be supported in a vertical position of an inclined position. Driving *hammers* are of the following types:

1. Drop hammer. A drop hammer is the one in which a hammer (or ram *or monkey*) is raised by winch and allowed to fall or drop by gravity on the top of the pile. The drop hammer is provided with lugs so that it can slide in the leads and a lifting eye or hook is provided to tie it with the rope. Ther weight of drop hammer varies from $\frac{1}{2}$ to 2 tonnes (5 to 20 kN) and the height of fall may vary from $1\frac{1}{2}$ to 3 metres. Fig. 4.20 shows the general arrangement. The number of blows that can be imparted varies from 4 to 8 per minute. Because of the slow speed, they have now become obsolete, except for piling short lengths.

2. Single acting hammer. If the hammer is raised by steam, compressed air or internal combustion, but is allowed to fall by gravity along, it is called a single acting hammer. The energy of such hammer is equal to the weight of the ram times the height

of fall. The weight of single acting hammer is about 2 tonnes (20 kN), the fall is about 1 metre and the number of blows of the hammer may vary from 50 to 60 per minute.

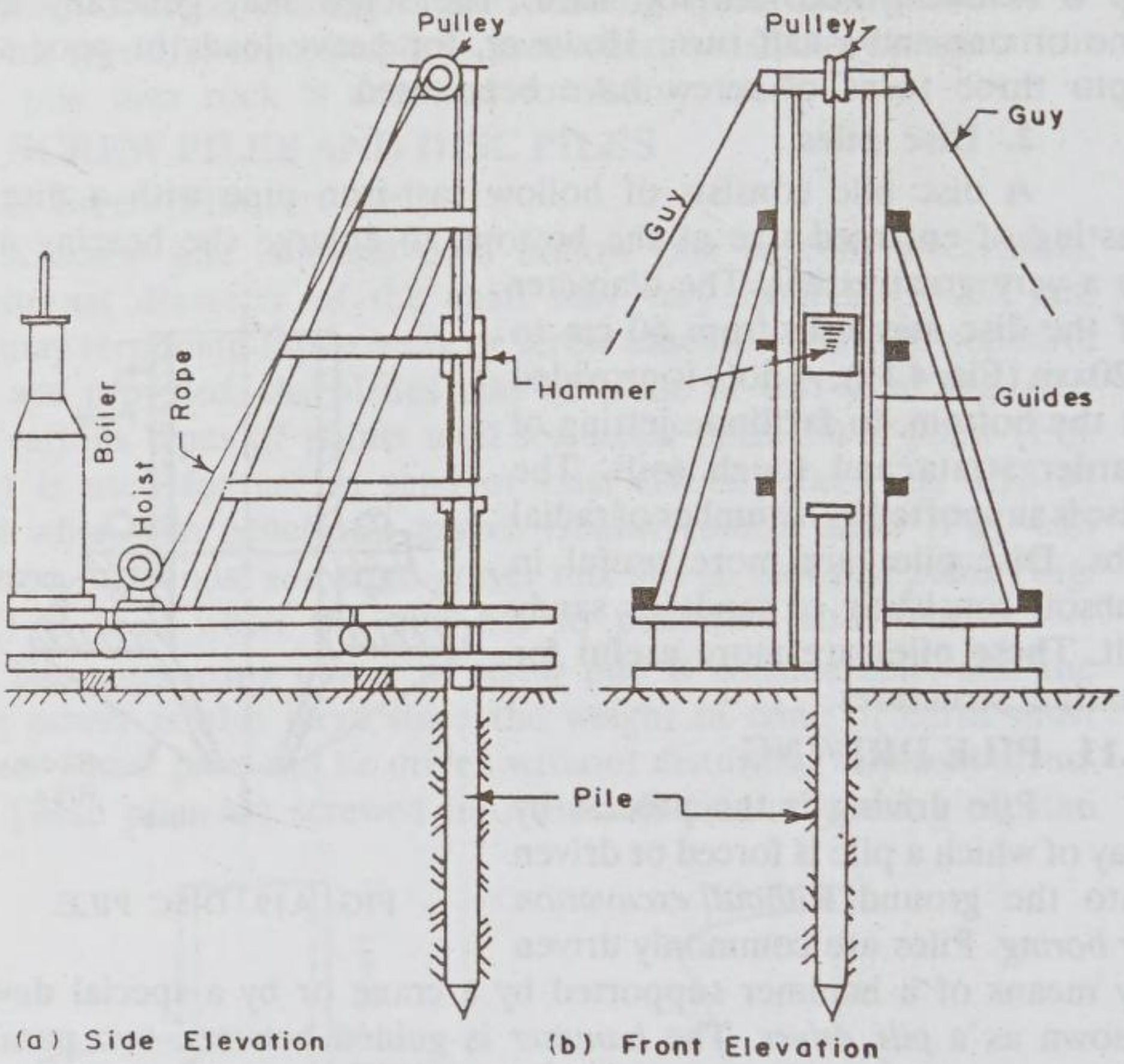


FIG. 4.20. PILE DRIVING WITH DROP HAMMER.

3. Double acting hammer. The double acting hammer employs steam or air for lifting the ram and for accelerating the downward stroke. It operates with succession of rapid blows, the number varying from 100 to 200 blows per minute. The weight of the hammer is only 500 kg (5 kN) but because of accelerating effect of steam(or air) pressure, it has an effect of a weight of 3 tonnes (30 kN). For light hammers, the number of blows may be even as high as 300 per minute. Because of such large number of blows, the pile drivind is very quick. The double-acting steam hammer is completely enclosed in a steel case. Therefore, these hammers are very useful for driving piles under water. Also, pile frame is not required, and the hammer is attacahed to the top of the pile by leg guides. A timber framework is provided to guide the pile. However, because of light weight of hammer, the equipment is not suitable for driving heavy piles through hard strata. In such cases, single acting hammers are generally used.

4. Diesel hammer. The diesel hammer is a small, light weight self-contained and self-acting type, using gasoline for fuel. The total driving energy is the sum of the impact of the ram plus the energy delivered by explosion.

5. Vibratory hammer. In this, the driving unit vibrates at high frequency.

Drive Cap or helmet. During pile driving, heads, helmets or caps are placed on the top of the pile to receive the blows of hammer and to prevent damage to the head of the pile. It is made of cast steel. It also helps in maintaining the axis of the pile in line with the axis of the hammer. The helmet is fitted with a timber stub dolly at its top. A cushion or pad of resilient material, saw dust, hard wood or rope is placed between the cap and the top of the pile to protect the pile head. Fig. 4.21 shows the details of the drive cap.

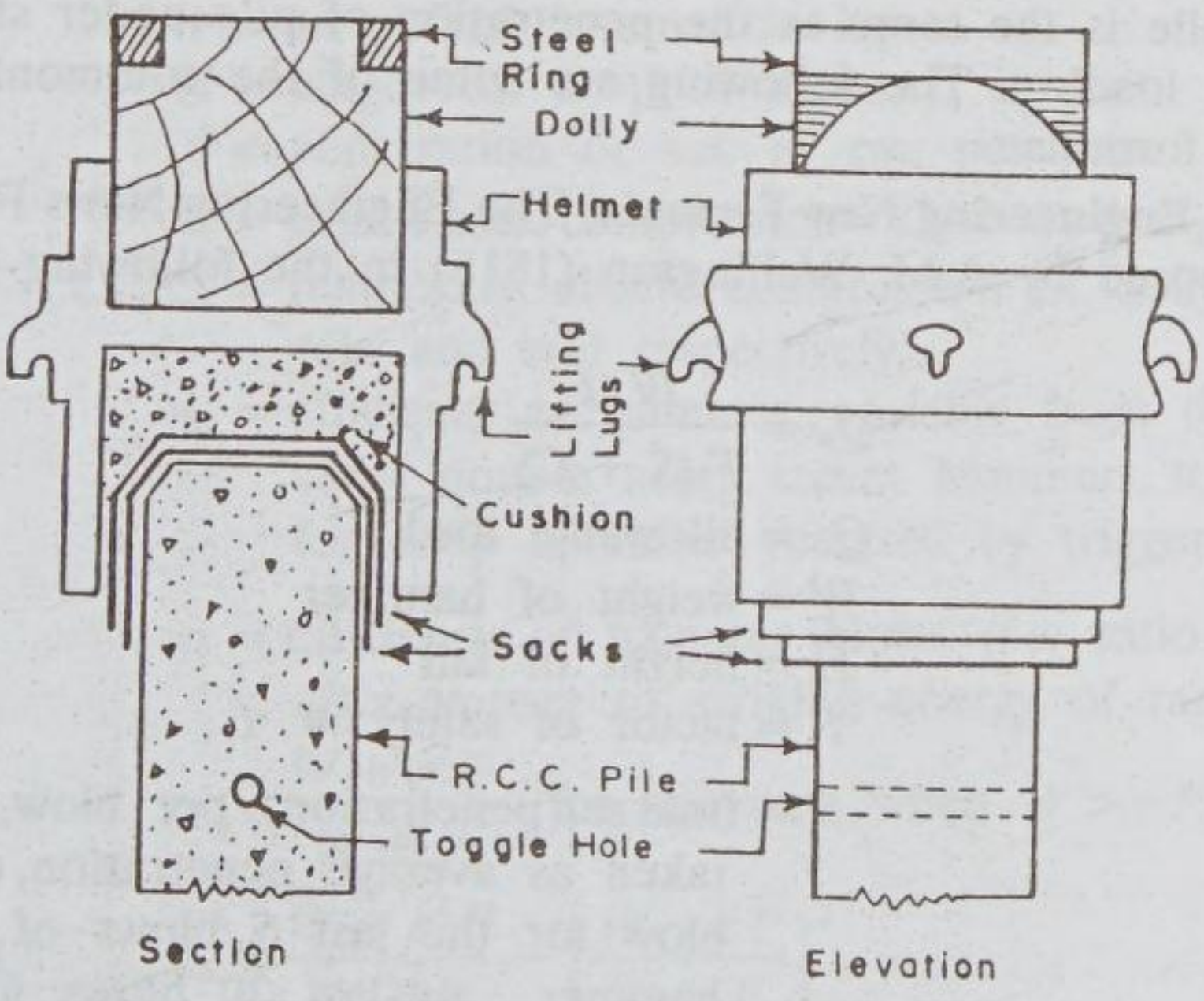


FIG. 4.21. PILE DRIVE-CAP OR HELMET.

Piles are ordinarily driven to a resistance measured by the number of blows required for the last 1 cm of penetration. Resistance of 3 to 5 blows per cm are commonly specified for concrete pile.

4.12. LOAD CARRYING CAPACITY OF PILES

The ultimate load carrying capacity, or ultimate bearing capacity or the *ultimate bearing resistance* Q_f of a pile is defined as the maximum load which can be carried by a pile and at which the pile continues to sink without further increase of load. The *allowable load* Q_a is the safe load which the pile can carry safely and is determined on the basis of (*i*) ultimate bearing, resistance divided by appropriate

factor of safety. (*ii*) the permissible settlement, and (*iii*) overall stability of the pile foundation . The load carrying capacity of a pile can be determined by the following methods.

1. Dynamic formulae
2. Static formulae
3. Pile load tests
4. Penetration tests.

(*a*) DYNAMIC FORMULAE

These are used for precast concrete piles. When a pile hammer hits the pile, the total driving energy is equal to the weight of hammer times the height of drop or stroke. In addition to this, in the case of double acting hammers, some energy is also imparted by the steam pressure during the return stroke. The total downward energy is consumed by the work done in penetrating the pile and by certain losses. The various dynamic formulae are essentially based on this assumption. It is also assumed that soil resistance to dynamic penetration of pile is the same as the penetration of pile under static or sustained loading. The following are some of the commonly used dynamic formulae.

(*i*) **Engineering New Formula.** The Engineering News Formula was proposed by A.M. Wellington (1818) in the following general form:

$$Q_a = \frac{WH}{F(S + C)} \qquad ...(4.1)$$

where Q_a = allowable load
W = weight of hammer
H = height of fall
F = factor of safety = 6
S = final set(penetration) per blow, usually taken as average penetration, cm per blow for the last 5 blows of a drop hammer , or last 20 blows for a steam hammer.
C = empirical constant.

Denoting W in kg, H in cm, and
C = 2.5 cm for drop hammers, and
= 0.25 cm for single or double acting hammers.

The above formula reduces to the following forms:

(*i*) *For Drop Hammers*

$$Q_a = \frac{WH}{6(S + 2.5)} \qquad ...(4.2)$$

(ii) For single acting steam hammers

$$Q_a = \frac{W H}{6\,(S + 0.25)} \qquad ...(4.3)$$

(iii) For double acting steam hammers

$$Q_a = \frac{(W + a\,p)\,H}{6\,(S + 0.25)} \qquad ...(4.4)$$

where a = effective area of piston (cm^2)

p = mean effective steam pressure (kg/cm^2)

2. Hiley's Formula (IS formula) Indian standard IS: 2911 (Part I) 1964 gives the following formula based on original expression by Hiley :

$$Q_f = \frac{\eta_h \,.\, W \,.\, H\, \eta_b}{S + \frac{C}{2}} \qquad ...(4.5)$$

where Q_f = ultimate load on pile.

W = weight of hammer in kg

H = height on drop of hammer, in cm

S = penetration or set, in cm, per blow

C = total elastic compression = $C_1 + C_2 + C_3$

C_1, C_2, C_3 = temporary elastic compression of dolly and packing, pile and soil respectively.

η_h = efficiency of hammer, variable from 65 percent for same double acing steam hammers to 100 percent for drop hammers released by trigger.

η_b = efficiency of hammer blows (*i.e.* ratio of the energy after impact to striking energy of ram).

$$\eta_b = \frac{W + e^2 P}{W + P} \text{ (for the case when } W > e\,P) \qquad ...(4.6)$$

and

$$\eta_b = \frac{W + e^2 P}{W + P} - \left\{\frac{W - e\,P}{W + P}\right\}^2,$$

(for the case when $W < e\,P$) ...(4.7)

P = weight of pile, helmet, follower.

e = co-efficient of restitution (variable from zero for a timber pile with poor cond ition of head or for excess packing in the driving cap to 0.5 for double acting hammer driven steel piles without driving cap or reinforced concrete piles without helmet but with packing on top.)

Eqs. 4.5, 4.6, and 4.7 are applicable for friction piles. For piles driven to refusal on rock (end bearing pile), a value of 0.5 P is substituted in the above expression. The product $\eta_h H$ is some

times referred to as the *effective fall* of the hammer.

For double acting hammers, the *rated energy* in the same length unit as S and C is substituted for WH. The allowable load is obtained by using a factor of safety of 2 or 2.5.

Comments about the use of dynamic formulae.

1. Dynamic formulae are best suited to coarse grained soils for which the shear strength is independent of rate of loading, because they allow to development of excess pore pressure around the pile during driving if saturated or dry.

2. The great objection to any one of the pile driving formulae is the uncertainty about the relationship between the dynamic and static resistance of soil.

3. In case of submerged loose uniform fine sands, impact of driving may cause liquefacion of soil, thus showing much less resistance than that which will occur under a static load. Similarly a very dense saturated fine sand may show an increased driving resistance which decreases with time.

4. For clays, the dynamic formulae are valueless because the skin friction developed in clay during driving is very much less (due to change in soil structure) than which occurs after a period of time. Also, the point resistance is much more at the time of driving because of pore pressure developed in clay, which reduces later on when the pore pressure dissipate.

5. Dynamic formulae give no indication about probable future settlement or temporary changes in soil structure.

6. The formulae do not take into account the reduced bearing capacity when in a group.

7. Law of impact used for determining energy loss is not strictly valid for piles subjected to restraining influence of the surrounding soil.

8. In Engineering News Formula, the weight of the pile and hence its inertia effect is neglected.

9. Energy losses due to vibrations, heat and damage to dolly or packing are not accounted for.

10. In Hiley's formula, a number of constants are involved, which are difficult to be determined.

(*b*) STATIC FORMULAE

The static formulae are based on the assumption that the ultimate bearing capacity Q_f of a pile is the sum of the total ultimate skin friction R_f and total ultimate point or end bearing resistance R_p :

$$Q_f = R_f + R_p$$

or

$$Q_f = A_s \,.\, r_f + A_p \,.\, r_p \qquad \text{...(4.6)}$$

where A_s = surface area of pile upon which the skin friction acts

A_p = area of cross-section of the pile on which bearing resistance acts. For tapered piles, A_p may be taken as the cross-sectional area at the lower one-third of the embedded length.

r_f = average unit skin friction, which may be taken equal to unit cohesion for cohesive soils.

r_p = unit point or toe resistance, which may be taken as $9c$ for cohesive soils.

Thus, for cohesive soils the above formula reduces to

$$Q_f = c\,A_f + 9\,c\,A_p \qquad ...(4.7)$$

A factor of safety of 3 may be adopted for finding the allowable load.

Example 4.1. *A wooden pile is being driven with a drop hammer weighing 2000 kg and having a free fall of 1.0 m. The penetration in the last blow is 5 mm. Determine the load carrying capacity of the pile according to Engineering News Formula.*

Solution. $$Q_a = \frac{W\,H}{6\,(S + C)}$$

where $W = 2000$ kg ; $H = 1 \times 100 = 100$ cm

$S = 0.5$ cm ; $C = 2.5$ cm.

$$Q_a = \frac{2000 \times 100}{6\,(0.5 + 2.5)} = \mathbf{11.1\ t}$$

Example 4.2. *A reinforced concrete pile, weighing 30 kN (inclusive of helmet and dolly) is driven by a drop hammer weighing 40 kN and having an effective fall of 0.8 cm. The average set per blow is 1.4 cm. The total temporary elastic compression is 1.8 cm. Assuming the co-efficient of restitution as 0.25 and a factor of safety of 2, determine the ultimate bearing capacity and allowable load for the pile.*

Solution. Given $P = 30$ kN and $W = 40$ kN

$\eta_h \,.\, H = 0.8$ m $= 80$ cm ;

$S = 1.4$ cm ; $C = 1.8$ cm

$e = 0.25$; $F = 2$

Since $W > e\,P$,

$$\eta_b = \frac{W + e^2 P}{W + P} = \frac{40 + 30\,(0.25)^2}{40 + 30}$$

$$= 0.597$$

$$Q_f = \frac{(\eta_h H)\, W \eta_b}{S + \frac{C}{2}}$$

$$= \frac{80 \times 40 \times 0.597}{1.4 + \frac{1.8}{2}} = 830 \text{ kN}$$

$$Q_a = \frac{Q_f}{F} = \frac{830}{2} = 415 \text{ kN}$$

4.13. PILE LOAD TEST

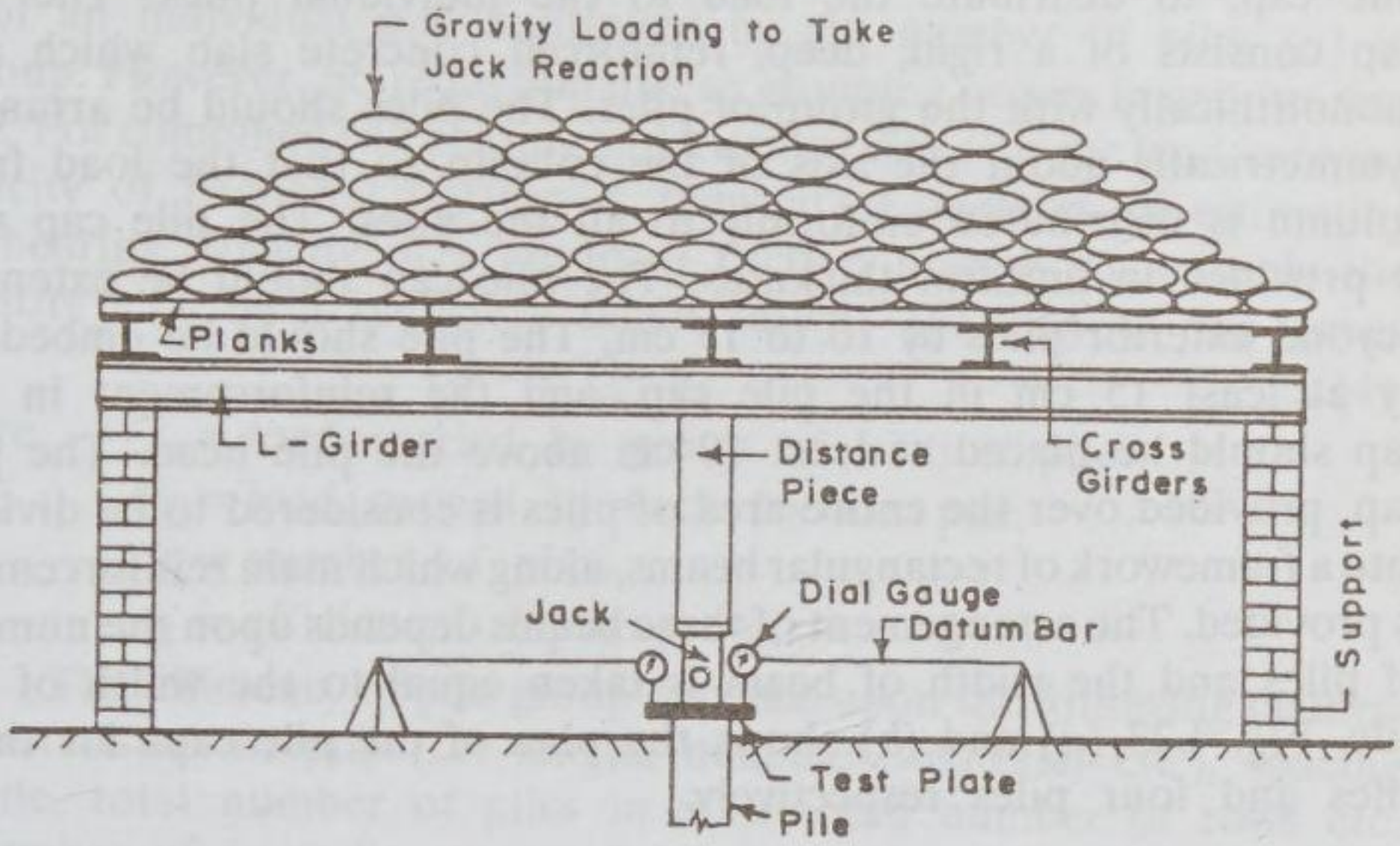
Pile load test is a reliable method of determining the carrying capacity of a pile. It can be performed either on a *working pile* which forms the foundation of the structure or on a *test pile.* The test load is applied with the help of a callibrated jack placed over a rigid circular or square plate which in turn is placed on the head of the pile projecting above ground level. The *reaction* of the jack is borne by a truss or platform which may have gravity loading in the form of sand bags etc. or alternatively, the truss can be anchord to the ground with the help of anchor piles. In the later case, under-reamed piles or *soil anchors* may be used for anchoring the truss. Both arrangements are shown in Fig. 4.22.

The load is applied in equal increments of about one-fifth of the estimated allowable load. The settlements are recorded with the help of *three* dial gauges of sensitivity 0.02 mm, symmetrically arranged over the test plate, and fixed to an independent datum bar. A remote controlled pumping unit may be used for the hydraulic jack. Each load increment is kept for sufficient time till the rate of settlement becomes less than 0.02 mm per hour. The test piles are loaded until ultimate load is reached. Ordinarily, the test load is increased to a value $2\frac{1}{2}$ times the estimated allowable load or to a load which causes a settlement equal to one-tenth of the pile diameter, whichever occurs earlier. The results are plotted in the form of load-settlement curve. The ultimate load is clearly indicated by the load-settlement curve approaching vertical. If the ultimate load cannot be obtained from the load settlement curve, the allowable load is taken as follows :

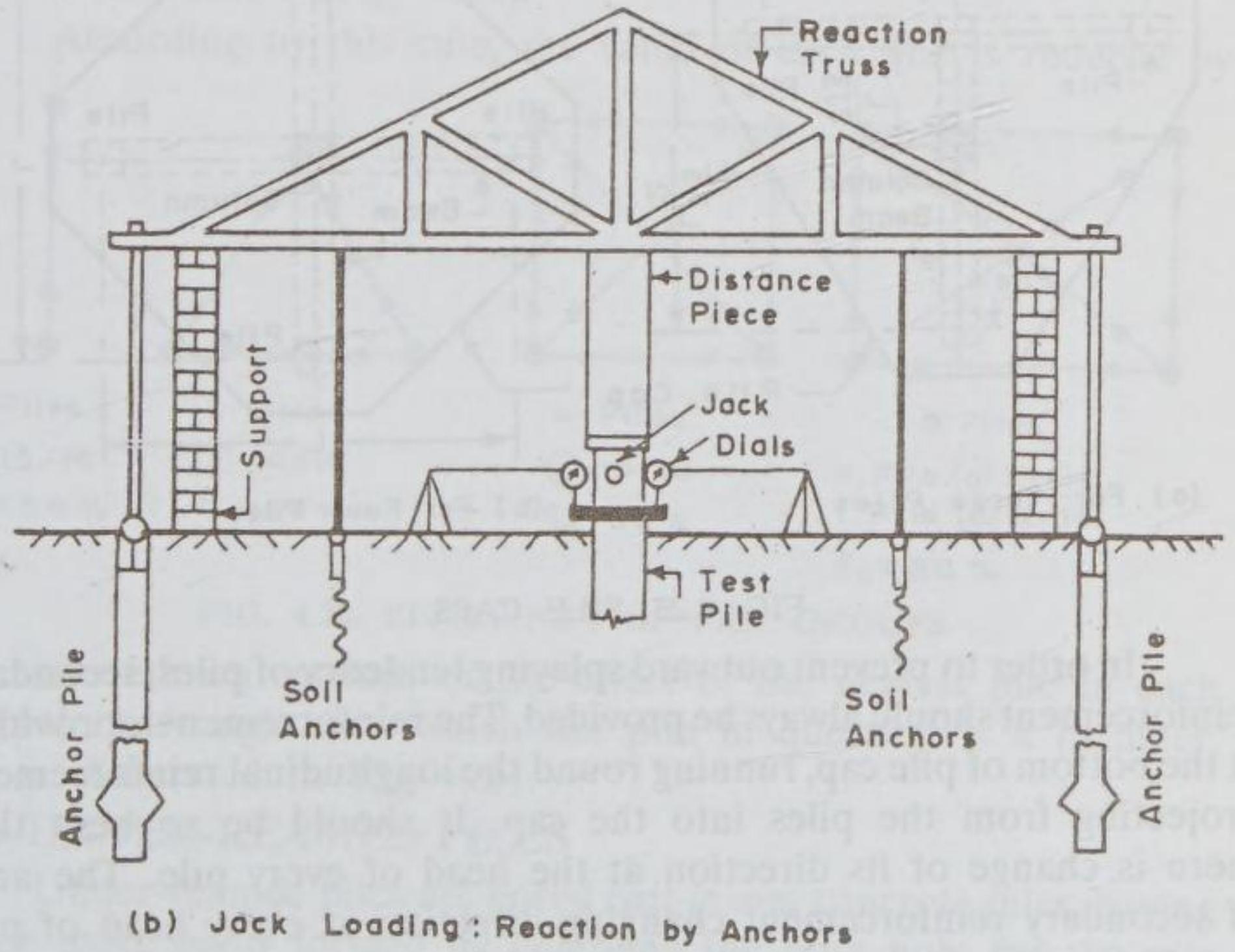
(*i*) One-half to one-third the final load which causes settlement equal to 10% of the pile diameter.

(*ii*) Two-thirds of the final load which causes a total settlement of 12 mm, or

(*iii*) Two-thirds of final load which causes a net settlement (residual settlement after the removal of load) of 6 mm.



(a) Jack Loading : Reaction by Loaded Platform



(b) Jack Loading : Reaction by Anchors

FIG. 4.22. ARRANGEMENTS FOR PILE LOAD TEST.

4.14. PILE CAP

When a column or pier is supported on one pile only, the column should rest centrally on pile. However, when the column or any other load carrying structural component is supported on more than one pile, the piles should be connected through a rigid

pile cap, to distribute the load to the individual piles. The pile cap consists of a rigid, deep, reinforced concrete slab which acts monolithically with the group of piles. The piles should be arranged symmetrically about the axis of the column so that the load from column is distributed uniformly to all the piles. The pile cap slab is provided in uniform thickness. The pile cap should be extended beyond exterior piles by 10 to 15 cm. The pile should be embedded by at least 15 cm in the pile cap, and the reinforcement in the cap should be placed at least 10 cm above the pile head. The pile cap, provided over the entire area of piles is considered to be divided into a framework of rectangular beams, along which main reinforcement is provided. The arrangement of these beams depends upon the number of piles and the width of beam is taken equal to the width of the pile. Fig. 4.23 (*a*) and (*b*) shows the plan of the pile caps for three piles and four piles respectively.

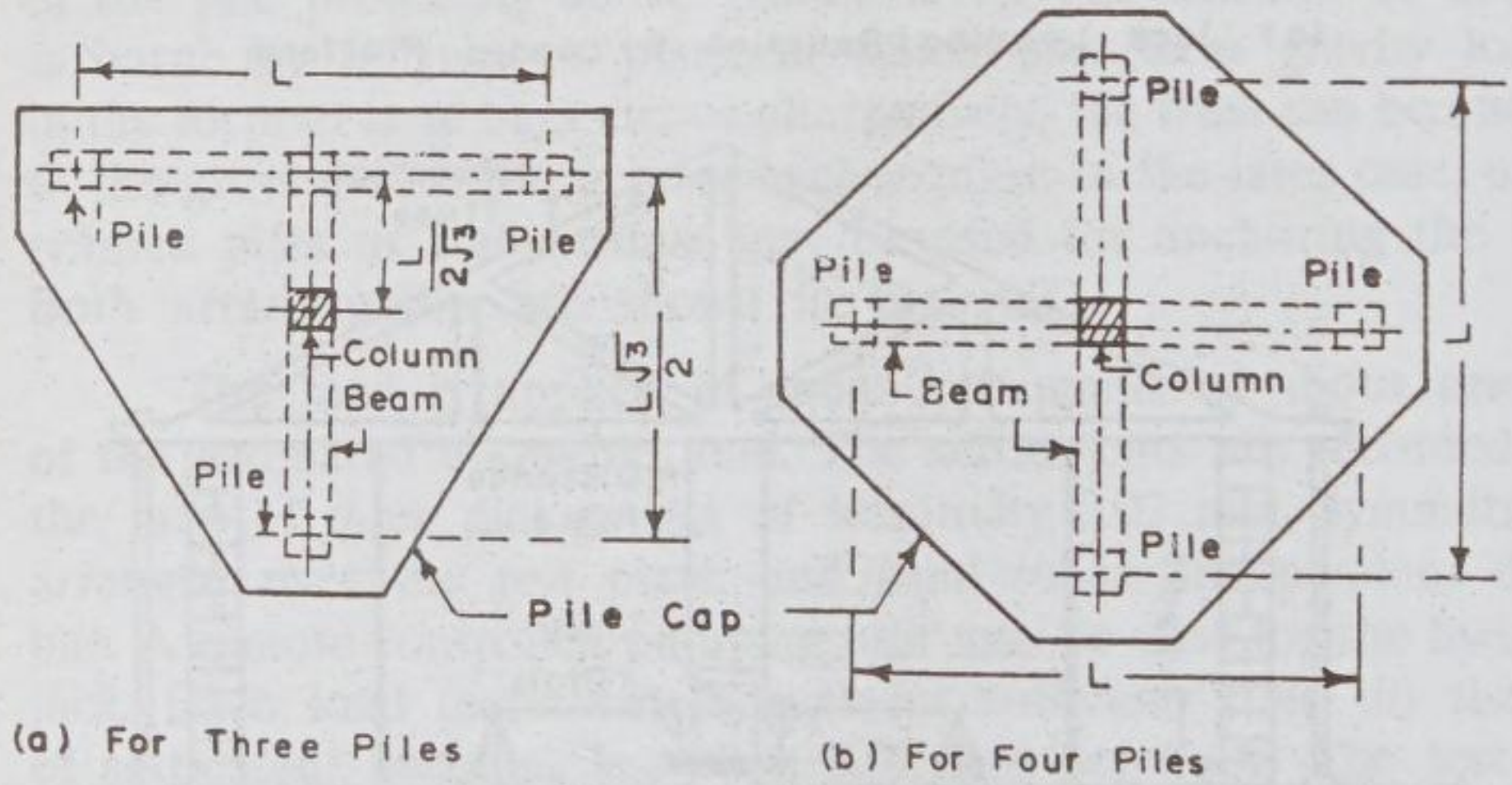


FIG. 4.23. PILE CAPS.

In order to prevent outward splaying tendency of piles, secondary reinforcement should always be provided. The reinforcement is provided at the bottom of pile cap, running round the longitudinal reinforcement projecting from the piles into the cap. It should be so bent that there is change of its direction at the head of every pile. The area of secondary reinforcement changing direction at every head of pile should not be less than 20% of the tensile reinforcement.

4.15. GROUP ACTION IN PILES

When several closely spaced piles are grouped together, it is reasonable to expect that the soil pressure developed in the soil as resistance will overlap. The bearing capacity of a pile group may or may not be equal to the sum of the bearing capacity of individual piles constituting a group. Theory and tests have shown that the total bearing value Q_g of a group of the friction piles, particularly

in clay, may be less than the product of the friction bearing value Q_f of an individual pile multiplied by the number of piles (n) in a group. However, no reduction due to grouping occurs in end bearing piles. For combined end bearing and friction piles, only the load-carrying capacity of frictional portion is reduced. A method of estimating the bearing capacity of a group of friction piles is to multiply the quantity $n\,Q_f$ by a reduction factor called *efficiency of pile group.*

$$Q_g = n\,Q_f \,.\, \eta_g \qquad ...(4)$$

where Q_g = load carried by group of friction piles,
Q_f = load carried by each friction pile,
n = number of piles,
η_g = efficiency of pile group.

The efficiency of pile group depends upon the following factors: characteristics of pile (*i.e.* length, diameter, material etc.), spacing of pile, total number of piles in a row and number of rows etc. A number of formulae are available for determining the efficiency of pile group. Out of these *Feld's* rule is given below.

Feld's Rule. (Fig. 4.24).

According to this rule, the value of each pile is reduced by

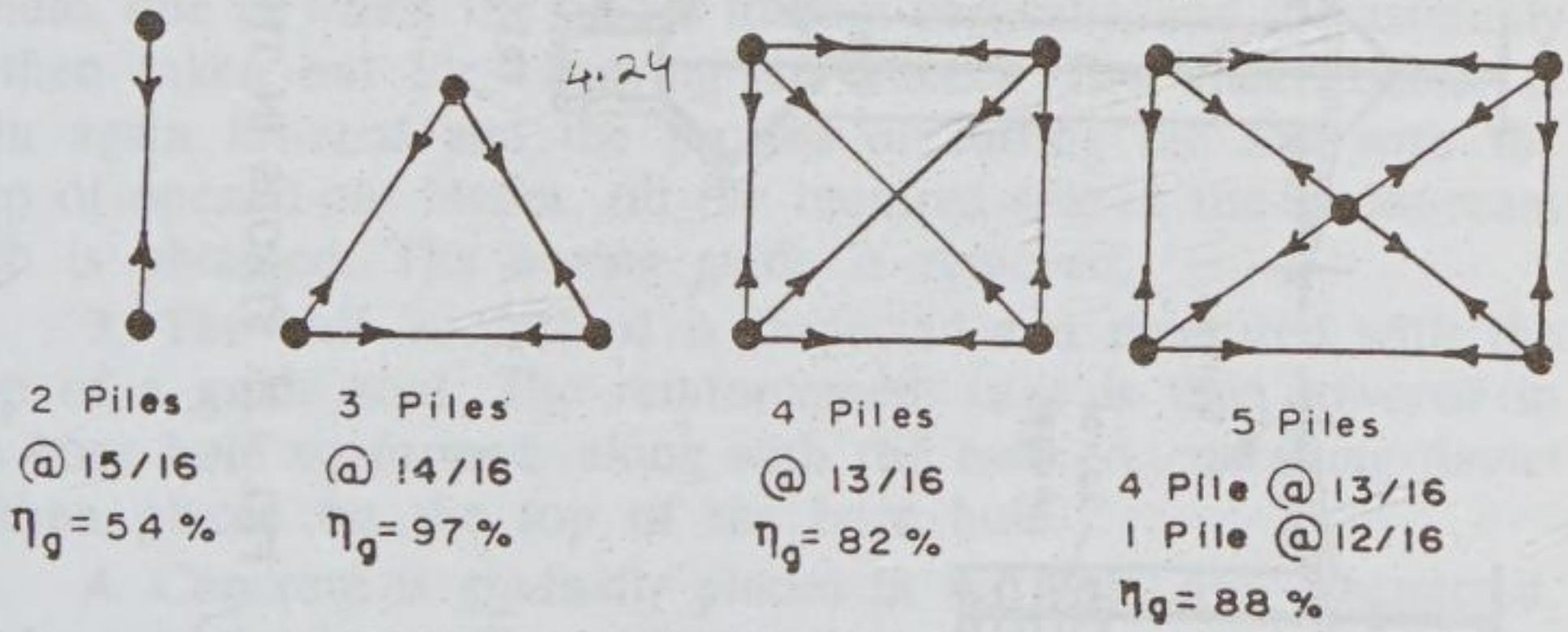


FIG. 4.24. EFFICIENCY OF PILE GROUPS.

one- sixteenth on account of the effect of the nearest pile in each diagonal or straight row which the pile in question is a member. This is illustrated in Fig. 4.24.

4.16. UNDER-REAMED PILES

Under-reamed piles are bored *cast-in-situ* concrete piles, having one or more *bulbs* formed by enlarging the bore hole for the pile stem by an under-reaming tool. These piles find applications in widely varying situations in different types of soils where foundations are required to be taken down to a certain depth to avoid the undesirable effects of seasonal moisture changes as in expansive soils or to obtain adequate capacity for downward, upward or lateral loads or to take the foundations below scour level and for moments.

Fig. 4.25. shows various stages in the formation of a under-reamed

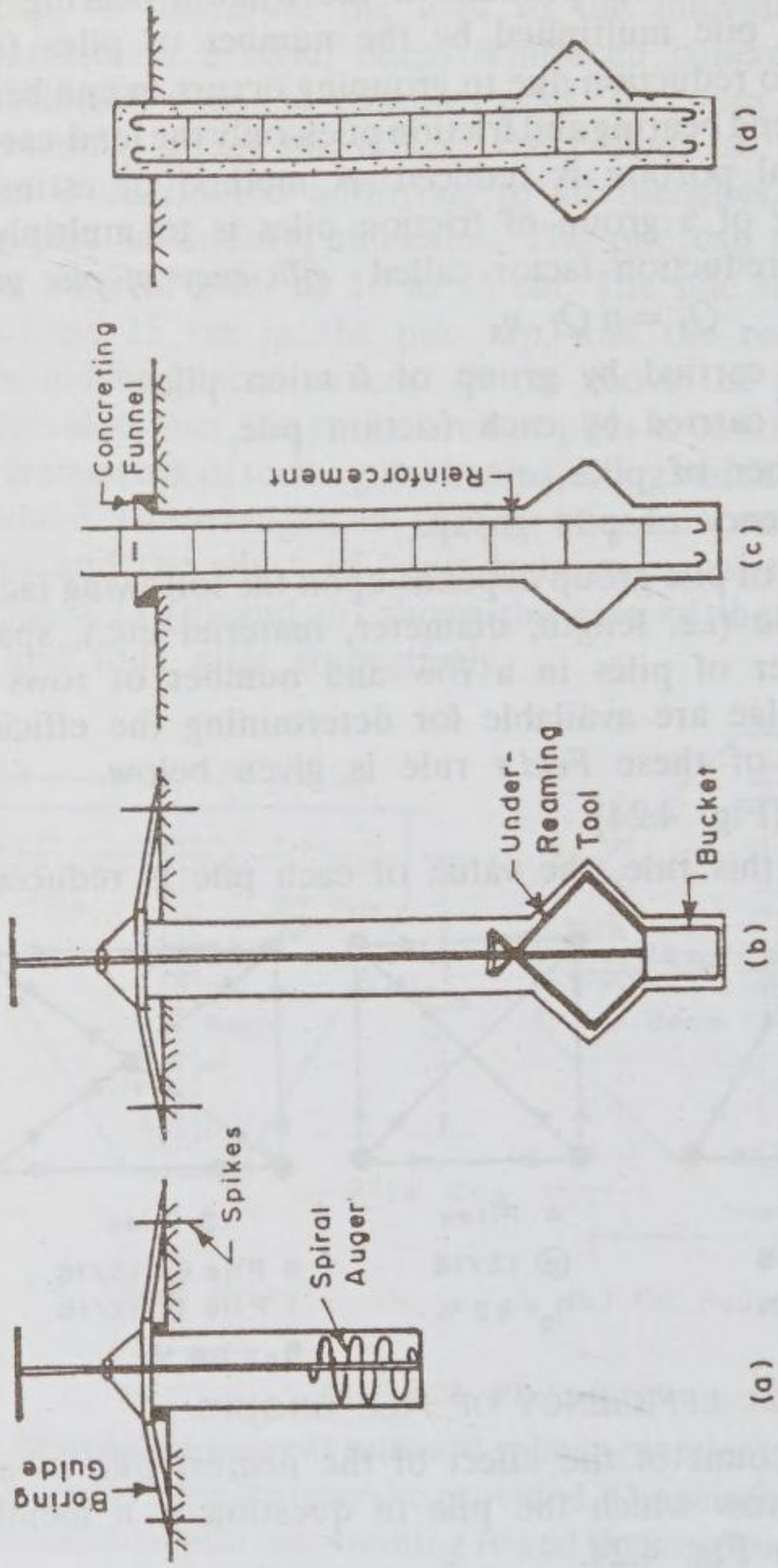


FIG. 4.25. STAGES IN THE CONSTRUCTION OF UNDER-REAMED PILE.

pile. The equipment required for the construction of pile are (*i*) auger boring guide (*ii*) spiral auger with extension rods (*iii*) under-reamer with soil bucket, and (*iv*) concreting funnel. A portable tripod hoist with whinch is required especially for piles longer than 4 to 5 m length and/or of diameter larger than 37.5 cm.

Procedure.

1. The ground is levelled and the boring guide is correctly positioned. The boring guide consists of a square frame with two

sets of flaps and four detachable arms having bolting arrangements at corners. Spikes are fixed, one in each arm. Soil inside the round collar is taken out. A spiral auger is lowered into the round hole so formed, and the flaps are tightened, thus encircling the vertical rod of the auger. The auger is then rotated, thus makingd a bore hole (Fig. 4.25 *a*), when the auger becomes full of soil, the flaps are loosened, and the auger full with soil is taken out. The auger is again lowered and the process repeated, till the desired depth is reached. Thus a straight vertical bore hole of the specified diameter is obtained. If the soil is not self-supporting, drilling mud may be suitably sprayed round the wall of the bore hole.

2. The under-reaming tool, attached with a bucket at its end is then lowered vertically down in the bore hole, with the help of the boring guide (Fig. 4.25 *b*). The *under-reaming too l*(or the under-reamer) consists of an assembly of two blades fixed around a central shaft and a detachable bucket for holding the cut soil. A pin inserted in the shaft controls the maximum diameter of the bulb to be cut. When pressure is applied on the lowered under-reamer assembly, the blades gradually widen or open out and cut the soil which drops in the bucket. When the bucket is full, a pull is applied to the handle, due to which the blades foldout vertically, and the assembly is then taken out for emptying the bucket. The under-reamer is then again lowered and the process of cutting the soil with the help of opened-out blades, till the required size of the under-ream bulb is obtained. The boring guide is removed.

3. The bulb so formed is inspected and measured with the help of a guide tool. The reinforcement cage is then lowered in the bore hole so formed, along with the bulb. A *concreting funnel* is then placed on the top of the bore hole.

4. Concrete is gradually placed in the hole, and compacted. In the initial stages of concreting, the reinforcement cage can be raised and owered in concrete. Fig. 4.25 (*d*) shows the final form of the under-reamed pile so obtained.

Details of pile, under-reamed bulb and grade beam.

When the pile has one bulb, it is known as *single under-reamed pile*, while the pile with more than one bulb is known as *multi-under-reamed* pile. Generally, the diameter of the under-reamed bulb is kept equal to $2\frac{1}{2}$ times the diameter of the pile stem. However, it may vary from 2 to 3 times the stem diameter, depending upon the design requirements and feasibility of construction. When more than one bulb is to be formed, the bore hole is excavated corresponding to the position of the top bulb, and then under-reaming is done. When the first bulb has been formed, the boring is continued further with the help of spiral auger, till the depth upto the second bulb

is reached. The second bulb is then formed with the help of under-reaming tool. The process is continued till the desired depth is reached.

In deep layers of expansive soils, the minimum length of pile required is 3.5 m where the ground movements become negligible. In shallow depths of expansive soils and other poor soils depending upon the load requirements, the length may be reduced and the piles may be taken upto at least 50 cm in stable zone(i.e. the zone where there are no ground movements due to seasonal moisture changes). The length may be increased for higher loads.

The diameter of manually bored piles range from 20 cm to 37.5 cm. The spacing of piles is considered in relation to the nature of the ground, the types of piles and the manner in which the piles transfer the loads to the ground. Generally, the centre to centre spacing for under-reamed piles should not be less than 2 D_u where D_u is the under-reamed diameter). It may be reduced to 1.5 D_u when a reduction in load carrying capacity of 10% should be allowed. For the spacing of 2 D_u, the bearing capacity of pile group may be taken equal to the number of piles multiplied by the bearing capacity of individual pile. If the adjacent piles are of different diameter, an average value for spacing should be taken. The maximum spacing of the under-reamed pile should not normally exceed $2\frac{1}{2}$ metres so as to avoid heavy capping beams. In buildings, the piles should generally be provided under all wall junctions to avoid point loads on beams. Positions of intermediate piles are then decided trying to keep the door opening fall in between two piles as far as possible.

In double and multi-under-reamed piles of size less than 30 cm dia., the centre to centre vertical spacing between the two under-reams may be kept equal to 1.5 D_u, while for piles of 30 cm or more, this distance may be reduced to 1.25 D_u. The upper bulb should not be placed too close to the ground. The minimum desirable depth of the centre of the bulb is 1.5 m or 2 D_u whichever is greater.

The under-reamed pile is normally reinforced with 10 to 12 mm dia. longitudinal bars and 6 mm of rings. The details of the reinforcement are shown in Table 4.1. A clear cover of 4 cm is provided.

The under-reamed piles are connected by a reinforced concrete beam, known as *capping beam or grade beam*. Fig. 4.26 shows the details of under-reamed pile foundation, along with the grade beam. For expansive soils, the grade beam is kept above the ground, with a clear air gap of 8 to 10 cm to provide space for the expansion(swelling) of the subsoil. In case of non-expansive soil however, mass concrete (1 : 3 : 6 or 1 : 4 : 8 mix) of 8 to 10 cm thickness is provided between the ground and the bottom of the beam, as shown in Fig. 4.26 (*d*). Fig. 4.26 (*c*) shows the details of the interior or exterior

beams for expansive soils. For interior beams, 50 mm thick concrete slab or brick on edge is provided on both the sides to cover the air gap. For exterior beam, the slab is provided to the inner face while the beam has a sharp edge (curtain wall) penetrating the ground to the outer face. Due to this the swelling soil can easily expand without exerting any swelling pressure on the beam.

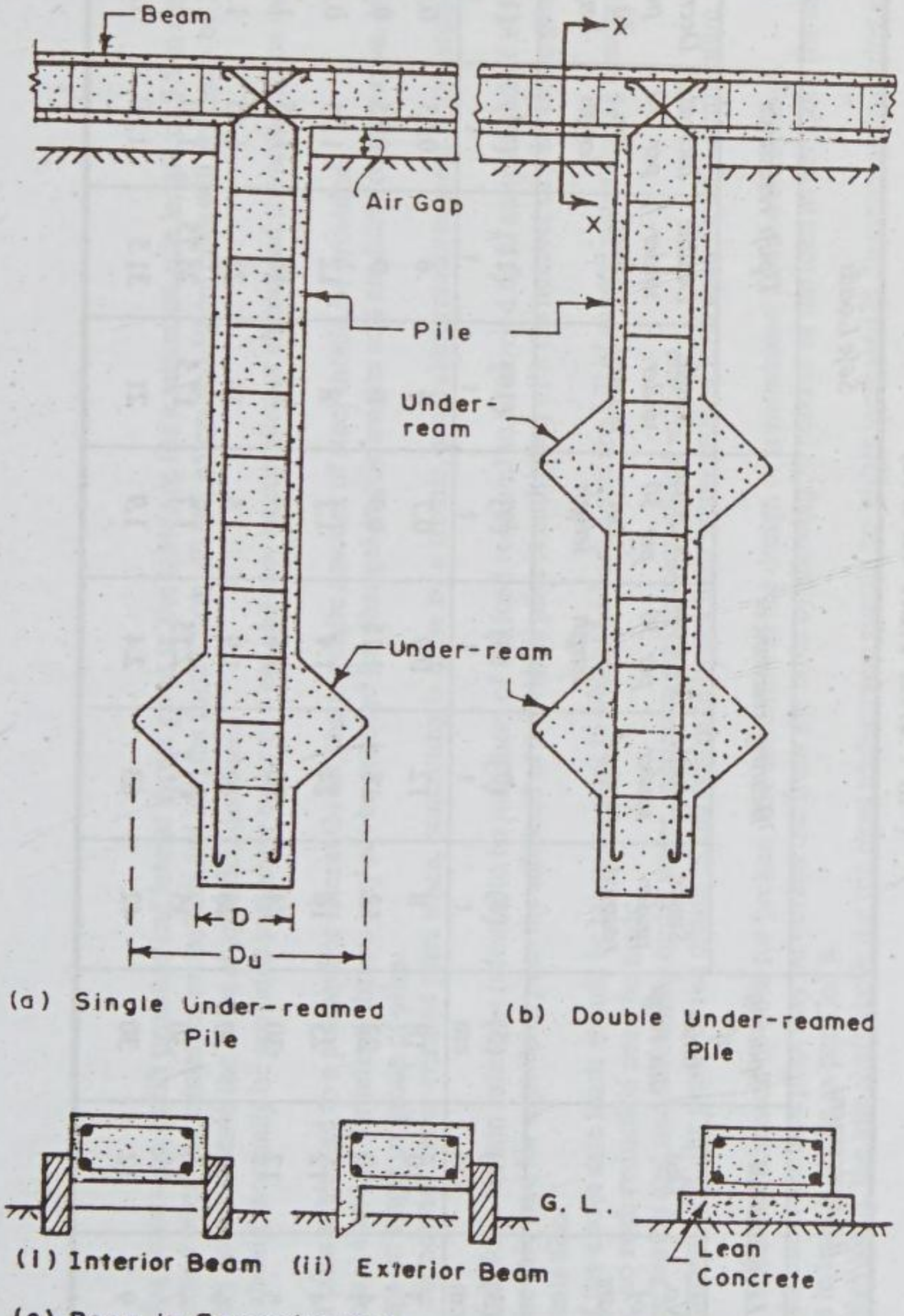


FIG. 4.26. DETAILS OF UNDER-REAMED PILE FOUNDATIONS.

Under-reamed piles can be made at a batter also for sustaining large lateral loads, thus making them suitable for tower footings, retaining walls and abutments. They have also been found useful

TABLE 4.1
SAFE LOADS FOR VERTICAL UNDER-REAMED PILES IN SANDY AND CLAYEY SOILS INCLUDING BLACK COTTON SOILS
(IS : 2911, PART III-1972)

Dia. of pile	*Under-reamed Dia. (D_u)*	*Reinforcement*			*Safe Loads*									
		Longitudinal		*Spacing of 6mm dia. rings*	*Bearing resistance*				*Uplift resistance*				*Lateral thrust*	
		No. of bars	*Dia.*		*Single under-reamed*	*Double under-reamed*	*Increase per 30 cm length*	*Decrease per 30 cm length*	*Single under-reamed*	*Double under-reamed*	*Increase per 30 cm length*	*Decrease per 30 cm length*	*Single under-reamed*	*Double under-reamed*
(1) cm	(2) cm	(3) cm	(4) cm	(5) cm	(6) t	(7) t	(8) t	(9) t	(10) t	(11) t	(12) t	(13) t	(14) t	(15) t
20	50	3	10	18	8	12	0.9	0.7	4	6	0.65	0.55	1.0	1.2
25	62.5	4	10	22	12	18	1.15	0.9	6	9	0.85	0.70	1.5	1.8
30	75	4	12	25	16	24	1.4	1.1	8	12	1.05	0.85	2.0	2.4
37.5	95	5	12	30	24	36	1.8	1.4	12	18	1.35	1.10	3.0	3.6
40	100	6	12	30	28	42	1.9	1.5	14	21	1.45	1.15	3.4	4.0
45	112.5	7	12	30	35	52.5	2.15	1.7	17.5	25.75	1.60	1.30	4.0	4.8
50	125	9	12	30	42	63	2.4	1.9	21	31.5	1.80	1.45	4.5	5.4

Note 1. The value of bearing resistance, uplift pressure and lateral thrust given in the table are for a minimum pile length of 3.5 m except in double under-reamed piles. In double under- reamed piles, the minimum recommended lengths for 37.5 cm, 40 cm, 45 cm and 50 cm piles will normally be 3.75 m, 4.0 m, 4.5 m and 5 m respectively so as to suitably accommodate the bulbs at specified distance.

Note 2. Longitudinal bars should normally be provided with clear cover of 4 cm and may be curtailed or eliminated towards the toe depending upon the stresses in pile section.

Note 3. For under-reamed piles subjected to a pull and/or lateral thrust, the requisite amount of steel should be provided.

Note 4. Values given in Cols. 14 and 15 for lateral thrusts may not be reduced for changes in pile lengths and are fairly conservative. Higher values may be adopted conducting lateral load tests on single or group of piles.

Note 5. In 25 and 30 cm. dia normal under-reamed piles when, concreting is done by a tremie, equivalent reinforcement in the shape of single iron piece placed centrally may be used.

Note 6. When a pile designed for a certain safe load is found to be just short of the load required to be carried by it, an overload of 10% should be allowed on it.

Note. 7. For working out the safe load for a group of piles the safe load of individual piles is multiplied with the number of piles in the group. This would be applicable for piles taking lateral thrusts also.

Note 8. Only 75 percent of the above safe loads should be taken for piles in which the bore holes are full of subsoil water during concreting. When water is confined to the bucket portion only, no such reduction need be made.

Note 9. In sandy soils when boring and under-reaming under water, minimum size recommended is 25 cm.

Note 10. In multi-under-reamed piles, the depth of the centre of the centre of upper bulb below ground level shall be kept a minimum of two times the diameter of under-ream bulbs.

Note 11. The values given should be increased by 50% for broken wire condition in the design of transmission line tower footings.

Note 12. Safe loads for multi-under-reamed piles may be worked out from the table by allowing 50 percent of the load as per Col.6 for each additional bulb. Increase in capacity due to increase in length will be as per Col. 8.

Note 13. For taking very high loads, the pile shaft above the top most under-ream should be either increased in diameter and/or additional reinforcement provided as in short column.

for factory buildings, machine foundations and transmission line towers and poles. In black cotton soils and other expansive soils, the under-reamed pile anchors the structure at a depth where the volumetric changes in soil due to seasonal and other variations is negligible.

Safe loads from IS Code Tables

The load carrying capacity of an under-reamed pile may be determined from load test. In the absence of actual load tests, the safe load allowed for piles under-reamed to 2.5 D_u may be taken from Table 4.1 based on IS : 2911, Part III-1975. The safe load given in the table apply to both medium compact sandy soil and clayey soils of medium consistancy. For dense sandy ($N \geq 30$) and stiff clayey ($N \geq 8$) soils the loads may be increased by 25%. However, the values of the lateral thrust should not be increased unless stability of the top soil (*i.e.*, strata to a depth of about 3 times the stem dia.) is ascertained. On the other hand, a 25 percent reduction should be made in case of loose sandy ($N \leq 10$) and soft clayey ($N \leq 4$) soils.

Load test on Under-reamed Piles

Piles are usually tested for determining the load-carrying capacity in compression, tension and lateral loading. Two categories of tests are conducted: (*a*) *initial tests* and (*b*) *routine tests*.. Initial tests should be carried out on test piles or working piles, but preferably on test piles. In case the initial tests show consistently higher or lower values than the estimated safe allowable loads on piles, designs should be re-examined and necessary modifications made. Routine tests are carried out at a check on working piles.

(*a*) ***Procedure for Initial Test (Compression)***

Following are the recommendations of Indian Standard IS: 2911 (Part III): 1973:

1. The test shall be carried out by applying a series of loads to the pile unaided by any other support. Pile groups may be tested as free standing piles or piled foundations as specified.

The load shall preferably be applied by means of hydraulic jack reacting against a loaded platform (Fig. 4.22 *a*) or rolled steel joists or suitable load frame held down by soil anchors and piles(Fig. 4.22 *b*) or other anchorage. The anchor piles may also be working piles but they shall be sufficient in number and adequately reinforced to take the full tension with proper factor of safety. The reaction available for loading should not be less than 3 times the estimated safe load-carrying capacity of piles. The jack should be of adequate capacity, preferably with a remote control pump and shall have pressure gauge or other suitable device for reading the applied load.

2. Readings of settlement shall be recorded with the help of at least 3 dial gauges of 0.02 mm sensitivity, positioned at equal

distances around the pile. The dial gauges shall be fixed to datum bars resting on non-movable supports at least $5D$ (Subject to a maximum of 2.5 m) away from the piles, where D is the pile stem diameter.

3. The test load shall be applied in increments of about 1/5 of the estimated safe load. At each stage of loading/unloading, the load shall be maintained till the movement of the pile top is not more than about 0.02 mm per hour.

4. Loading shall generally be continued upto $2\frac{1}{2}$ times the estimated safe load or to a settlement of 7.5% of the bulb diameter, whichever is earlier.

5. The *safe load* on pile shall be the least of the following:

(*i*) Two-thirds of the final load at which the total settlement attains a value of 12 mm, unless it is established that a total settlement different from 12 mm is permissible in a given case on the basis of nature and type of the structure; in the latter case the actual total settlement permissible shall be used for assessing the safe load instead of 12 mm.

(*ii*) 50 percent of the final load at which the total settlement equals 7.5 percent on the bulb diameter.

(*b*) *Proceudre for Routine Test (Compression)*

Loading shall be carried out upto $1\frac{1}{2}$ times the allowable load; the reaction provided may be 2 times the allowable load. The procedure followed for the test and determination of the allowable load shall be same as per initial test excepting clause 5(*ii*).

4.17. BORED COMPACTION PILES

Bored compaction piles are the modification of under-reamed pile. These piles are cast-in-situ piles which combine the advantages of both bored and driven pile. The method of boring the piles and concreting the pile is the same as that for the under-reamed pile, except that the reinforcement cage is not placed in the bore hole before concreting. After the concreting is over, the reinforcement cage is *driven* through the freshly laid concrete. Due to this feature, the compaction of surrounding soil as well as concrete are effected and the load-carrying capacity is increased by 1.5 to 2 times over normal under-reamed piles. These piles are particularly suitable in loose to medium dense sandy and silty strata. Also in cases of loose strata, overlying the dense strata specially in submerged soils, these piles can be sued with advantage. In such conditions, it is difficult to reach the desired depth in the case of bored piles normally without loosening the strata at pile toe.

Fig. 4.27 illustrates the stages of construction of such pile.

Procedure

1. Prepare the bore hole with the help of spiral auger, using

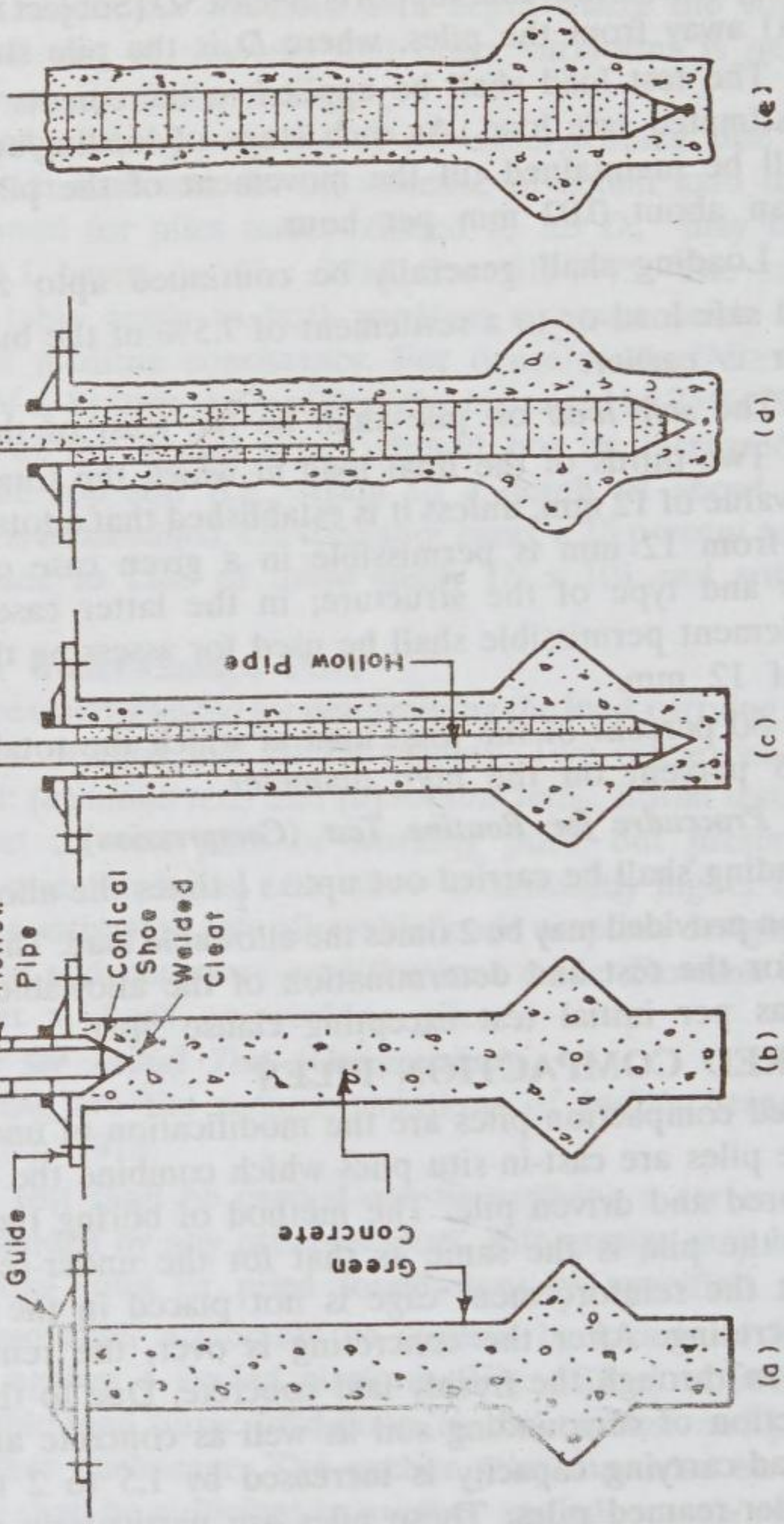


FIG. 4.27. CONSTRUCTION OF BORED COMPACTION PILE.

guides, and then under-ream it with the help of under-reaming tool, as is done for under-reamed pile. Concrete the pile, *without* placing the reinforcement cage (Fig. 4.27*a*).

2. Place of reinforcement cage, enclosing a hollow driving pipe, on the top of freshly laid concrete (Fig. 4.27 *a*). A cast iron conical shoe, with a iron cleat welded to it, attached to the reinforcing cage.

3. Drive the driving assembly through the freshly laid concrete to the full depth (Fig. 4.27*c*) by means of suitable drop weight (about 500 kg), operated with the help of mechanical winches. The movement of hammer and assembly should be controlled by suitable guiding attachment, to ensure vertical penetration of the cage. As the cage is driven into the concrete, soil and concrete gets compacted. This would result in increase in the diameter of the bore hole. Extra concrete is simultaneously poured to keep it level with the ground.

4. After driving through the full depth of concrete, fill concrete in the hollow drive pipe also. The pipe is then gradually withdrawn (Fig. 4.27 *d*) leaving the cage and concrete behind.

5. Fig. 4.27*e* shows the completed pile.

4.18. SAND PILES

Sand piles are usually compaction piles. A bore of required diameter, usually 20 cm to 40 cm, is formed either by an earth auger (*i.e.*, by boring) or by forcing a pipe with closed end. The hole so formed is then filled with sand which is well rammed. In case the hole is formed using casing tube, the tube is gradually withdrawn, leaving behind its flat end shoe. Water is added to sand while compacting it. The top of the sand pile is filled with concrete, to prevent the upward movement of sand due to lateral pressure. Sand piles may be 2 to 5 m deep, with top 1m filled with concrete (Fig. 4.28. Sand piles resting on firm strata can take loads of 100 tonne per m^2 or more. Thus a 30 cm dia. sand pile can take a load of 9 tonnes (90 kN).

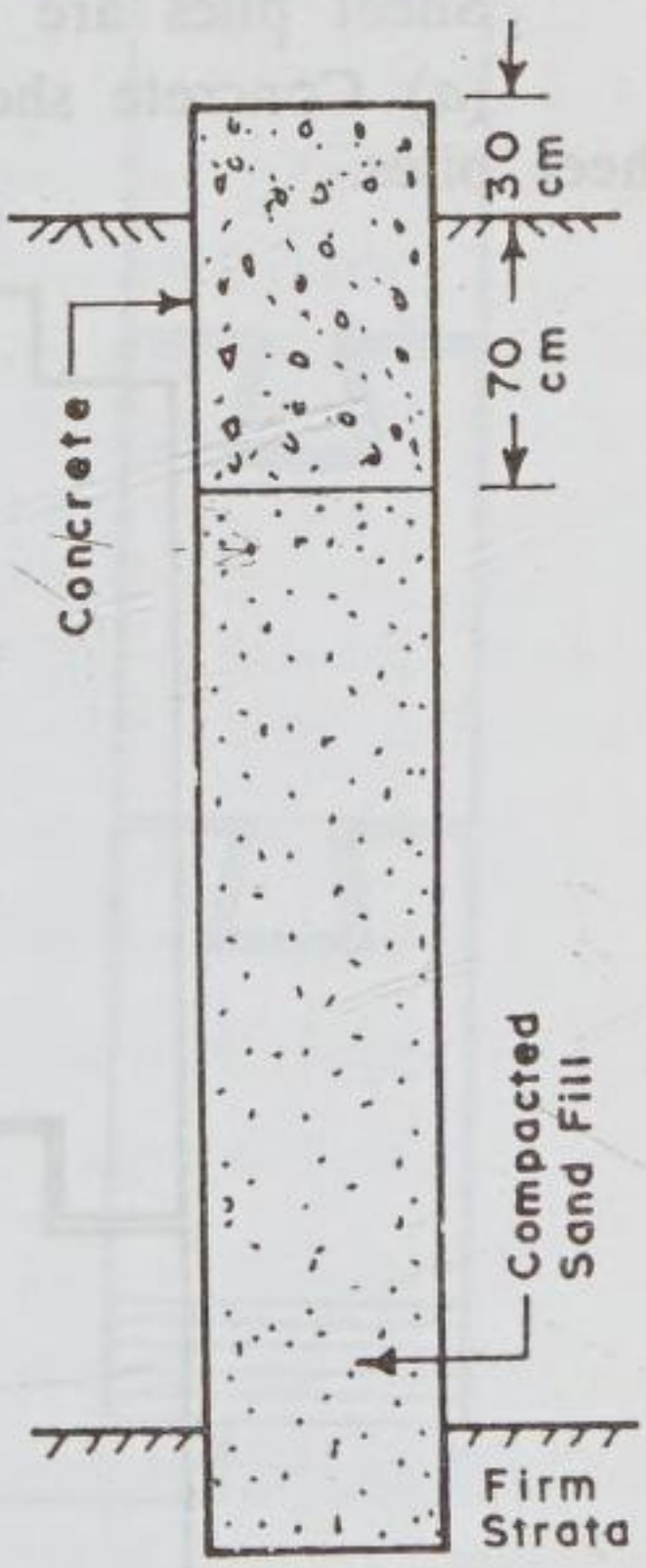


FIG. 4.28. SAND PILES.

4.19. SHEET PILES

Sheet piles are thin piles, made of plates of concrete, timber or steel, driven into the ground for either separating members or for stopping seepage of water. They are *not* meant for carrying any vertical load. They are driven into the ground with the help of suitable pile driving equipment,and their height is increased while driving, by means of addition of successive instalments of sheets.

Functions of Sheet piles. Sheet piles are used for the following purposes:

1. To enclose a site or part thereof to prevent the escape

of loose subsoil, such as sand, and to safeguard against settlement.

2. To retain the sides of the trenches and general excavation.

3. To form water tight enclosure (known as *coffer dam*) necessary in the construction of foundations in water.

4. To construct retaining walls in docks, wharfs and other marine structures.

5. To protect river banks.

6. To prevent seepage below dams and other hydraulic structures.

7. To confine the soil, thereby increasing the bearing capacity of soils.

8. To construct coastal defence works (as a protection against sea erosion).

9. To protect the foundations from scouring actions of nearby river, stream etc.

Sheet piles are made of the following materials :

(*a*) Concrete sheet piles, (*b*) Timber sheet piles, (*c*) Steel sheet piles.

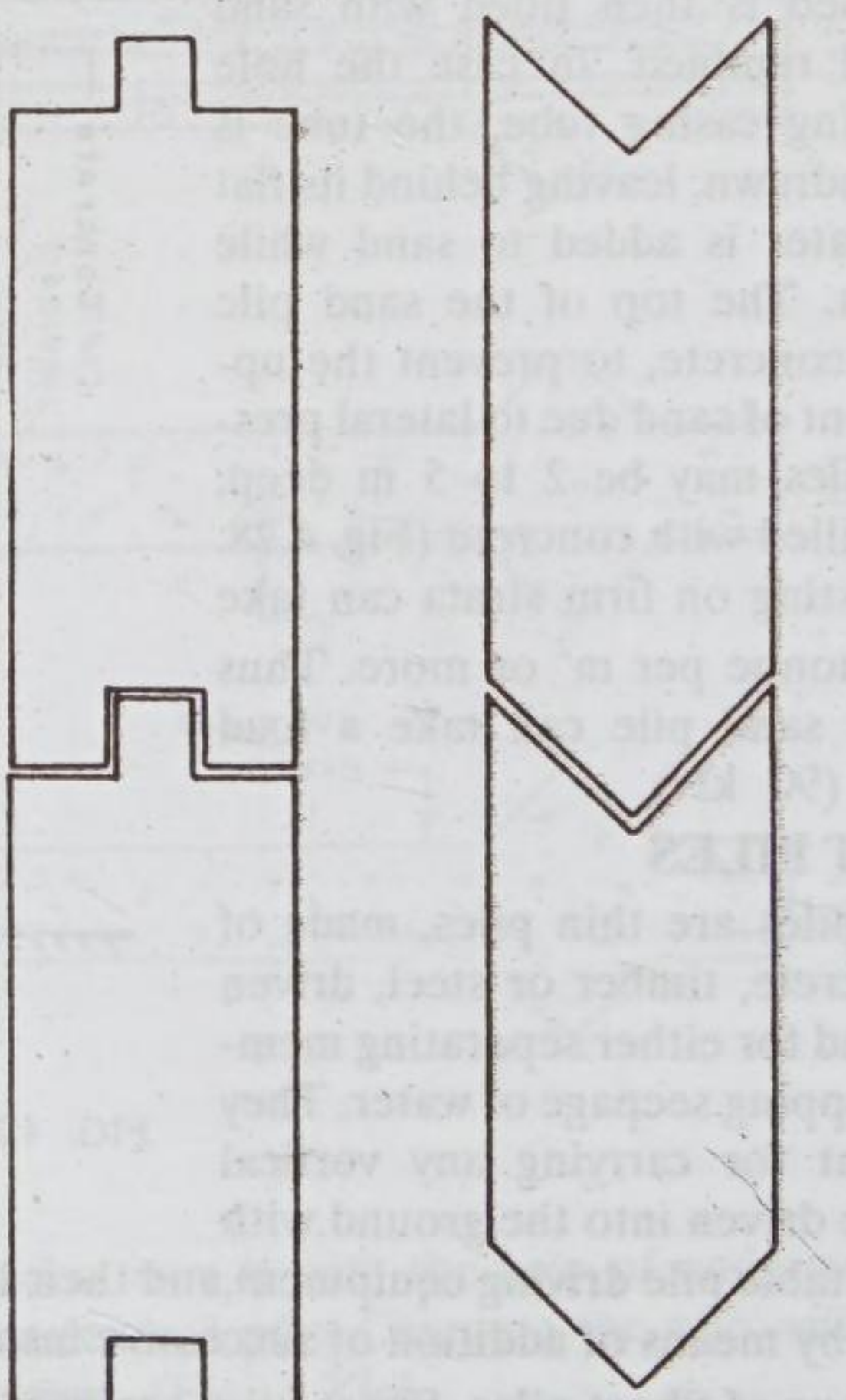


FIG. 4.29. PRE-CAST R.C.C. SHEET PILES

(*a*) Concrete sheet piles

Concrete sheet piles are reinforced, precast units. The width of each unit may vary from 50 cm to 60 cm and thickness varies from 2 cm to 6 cm. Fig. 4.29 shows typical sheet piles with proper jointing arrangements. For important works, pre-stressed pre-cast concrete sheet piles are used.

(*b*) Timber sheet piles

These are used only for temporary work. The width of the sheet may vary from 225 to 280 mm, while thickness should not be less than 50 mm. They may be jointed by either butt or V-joints. Their feet are bevelled, and some times shod with sheet iron.

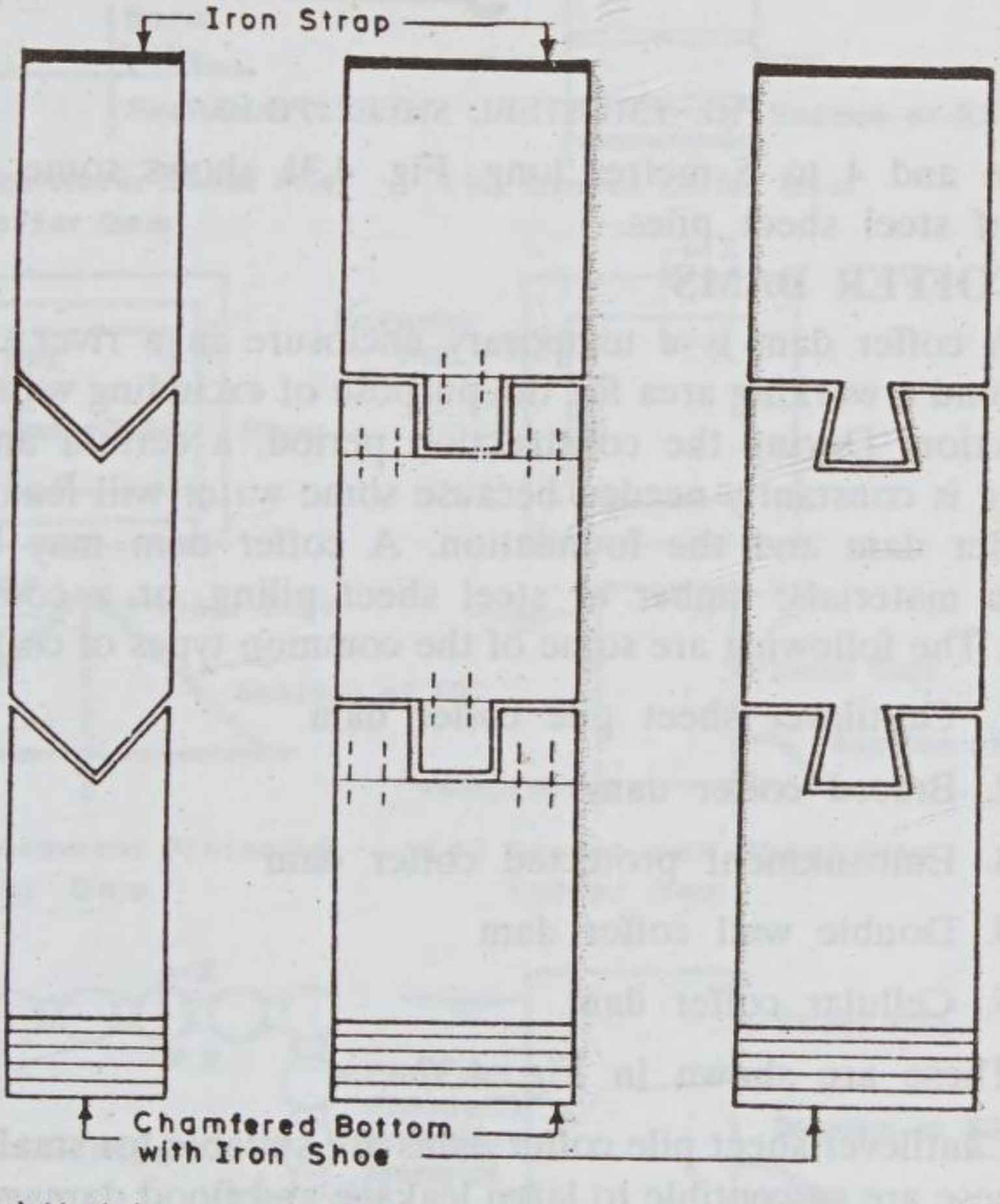


FIG. 4.30. TIMBER SHEET PILES.

(c) Steel sheet piles.

Steel sheet piles are most commonly used. They are trough shaped and, when driven, the piles are interlocked with alternate ones reversed. Sheet piles are available in different shapes, under different trade names. These are made from steel sheets 20 to 30

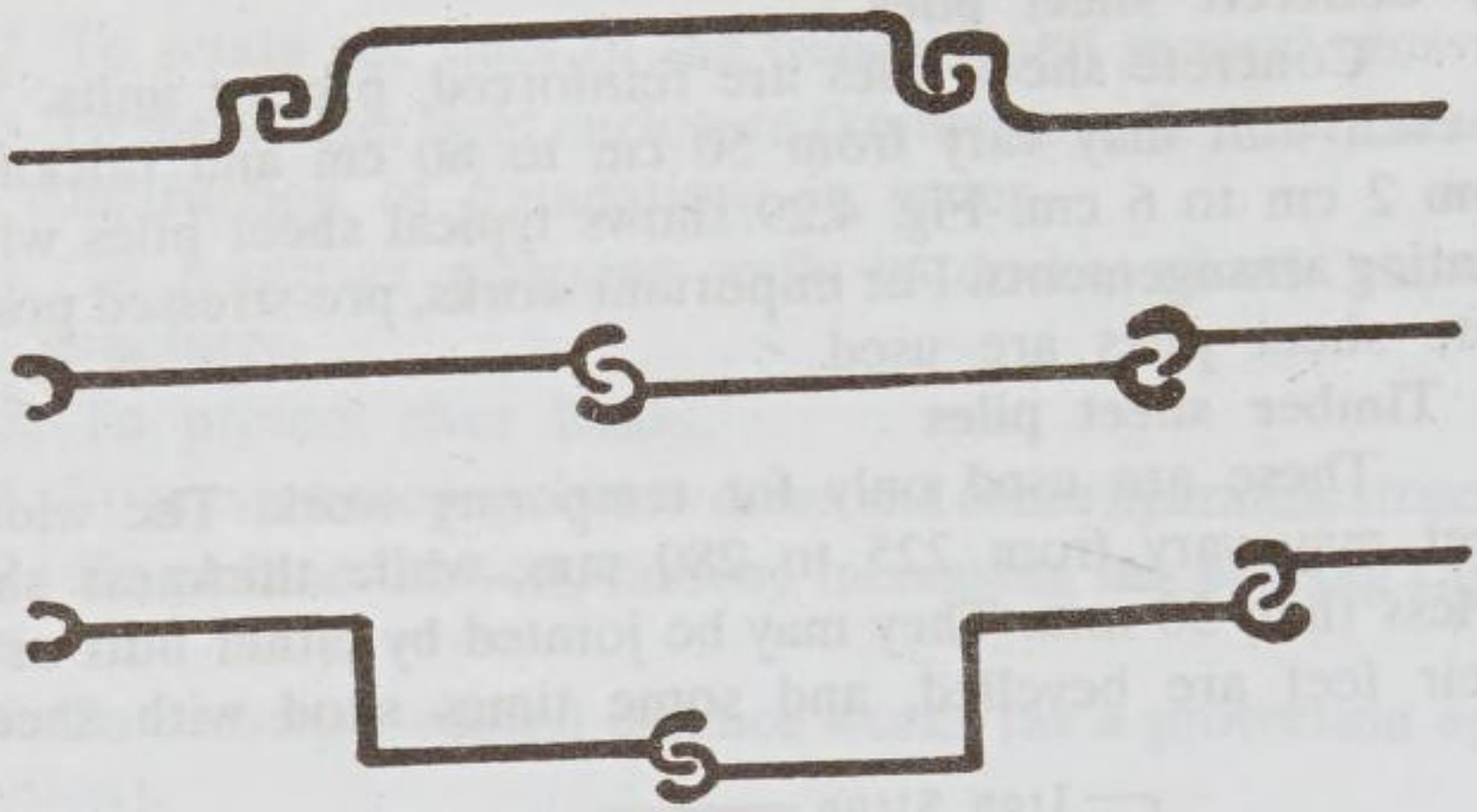


FIG. 4.31. STEEL SHEET PILES.

cm wide and 4 to 5 metres long. Fig. 4.31 shows some common forms of steel sheet piles.

4.20. COFFER DAMS

A coffer dam is a temporary enclosure in a river, lake etc. built round a working area for the purpose of excluding water during construction. During the construction period, a certain amount of pumping is constantly needed because some water will leak through the coffer dam and the foundation. A coffer dam may be made of earth materials, timber or steel sheet piling, or a combination thereof. The following are some of the common types of coffer dams:

1. Cantilever sheet pile coffer dam
2. Braced coffer dam
3. Embankment protected coffer dam
4. Double wall coffer dam
5. Cellular coffer dam.

These are shown in Fig. 4.32.

Cantilever sheet pile coffer dams are suitable for small heights, since these are susceptible to large leakage and flood damage. Braced coffer dams are economical for small to moderate height. For earth embankment type coffer dams, there is no height limitation, but since they occupy large base area, they are adopted only when the area to be excavated is very large. Double wall coffer dams are suitable for moderate height, while cellular coffer dams are suitable for moderate and large heights. A cellular coffer dam consists of a series of adjoining cells of circular or other curved shape, made of sheet piling. Each cell is huge vertical cylinder, 9 to 12 m in lateral dimension, and is filled with rock gravel and sand.

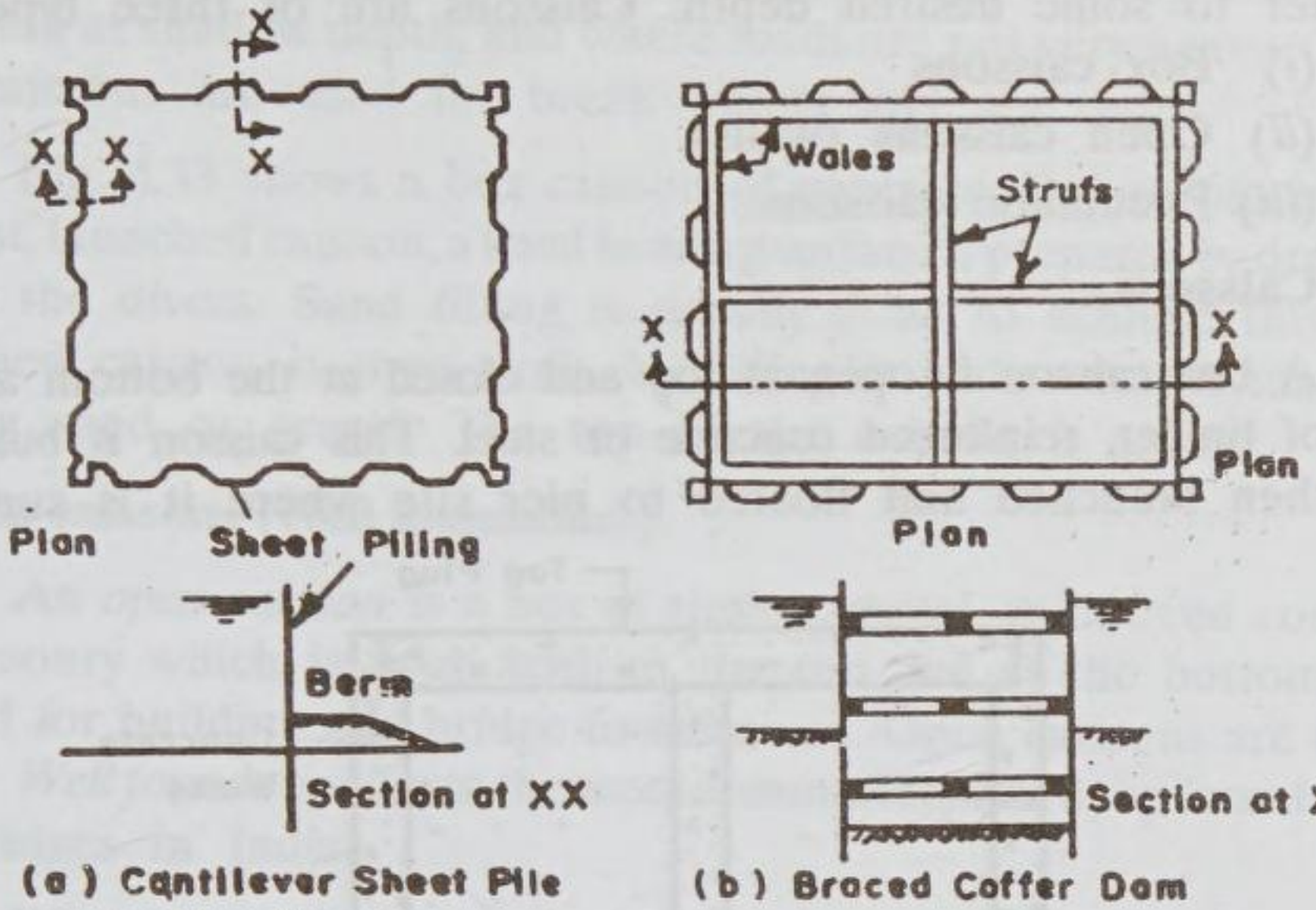


FIG. 4.32. COMMON TYPES OF COFFER DAMS.

4.21. CAISSONS : WELL FOUNDATIONS

The term 'Caisson' is derived from French word, *caisse* meaning a chest or box. Caisson has come to mean a box like structure, round or rectangular, which is sunk from the surface of either land

or water to some desired depth. Caissons are of three types:

(*i*) Box caissons
(*ii*) Open caissons (wells)
(*iii*) Pneumatic caissons.

1. Box Caissons

A box caisson is open at top and closed at the bottom and is made of timber, reinforced concrete or steel. This caisson is built on land, then launched and floated to pier site where it is sunk in

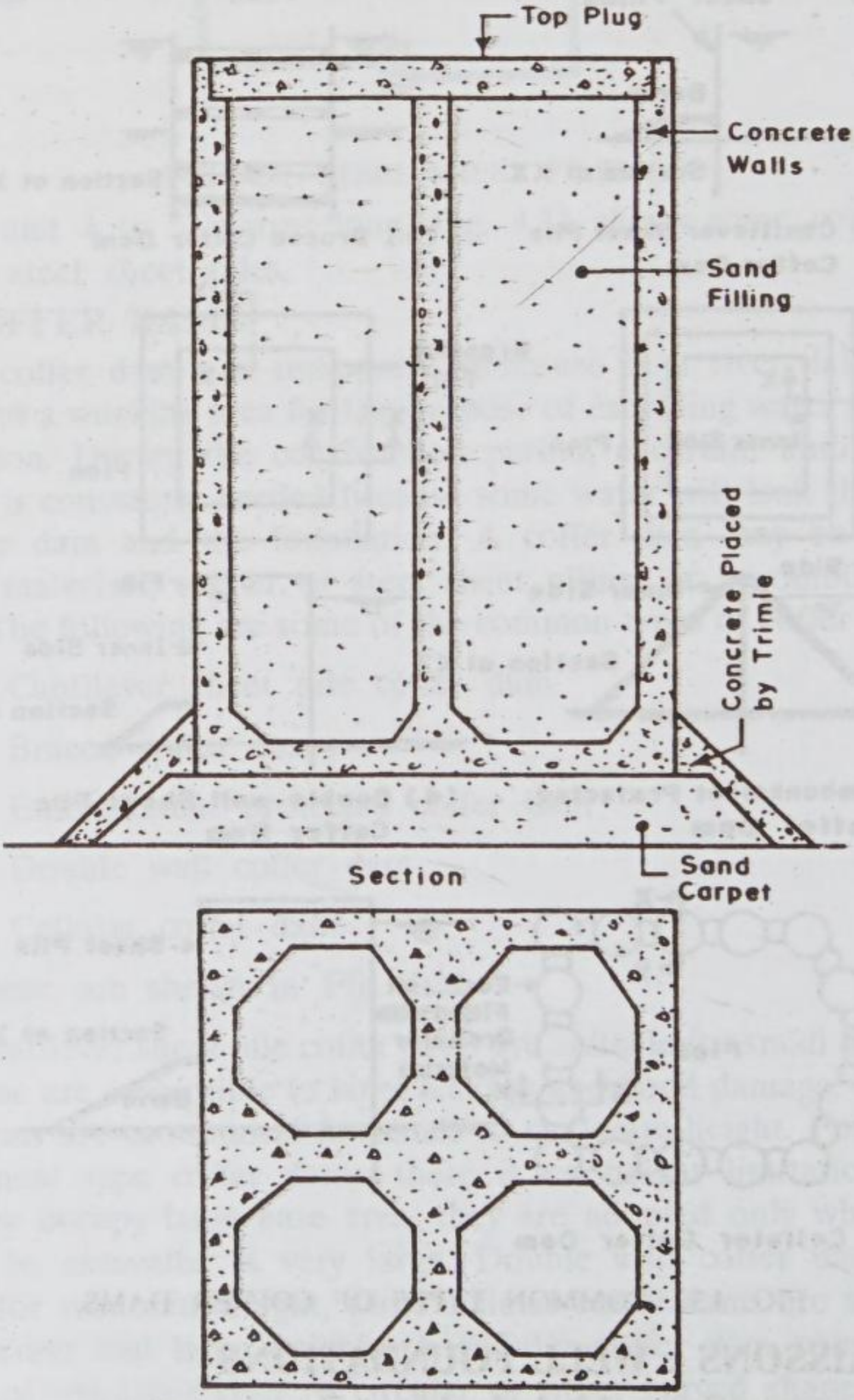


FIG. 4.33. BOX CAISSON (CONCRETE).

position. Such a type of caisson is used where bearing stratum is available at shailow depth, and where loads are not very heavy. Closed box caissons are used for break waters and sea walls.

Fig. 4.33 shows a box caisson of concrete. Before placing the precast, launched caisson, a level bearing surface is prepared by dredging or by the divers. Sand filling is usually done to achieve this. The launched caisson is then sunk, by filling it with suitable material, usually sand or gravel. The top of the caisson is sealed.

2. Open caissons (Well foundation).

An *open caisson* is a box of timber, metal, reinforced concrete or masonry which is open both at the top and at the bottom, and is used for building and bridge foundations. Open caissons are called *wells*. *Well foundation* form the most common type of deep foundations for bridges in India.

Whenever considerations for scour or bearing capacity require foundations being taken to a depth of more than 5 to 7 metres, open excavations become costly and uneconomical, as neavy timbering has to be provided. Also, because of the greater earthwork involved due to side slopes, the progress of work in open excavation will be very slow. Another disadvantage in adopting the ordinary type of footing is that excavated material refilled around the structure is loose and hence easily scourable as compared to natural ground. The above disadvantages are avoided in a well foundation which is a shell sunk by dredging inside of it and which finally becomes a part of the permanent structure.

Shapes of wells and component parts

The common types of well shapes are as follows (Fig. 4.34):

(1) Single circular
(2) Twin circular
(3) Dumb well
(4) Double-D
(5) Twin-hexagonal
(6) Twin-octagonal
(7) Rectangular.

The choice of a particular shape depends upon the dimensions of the base of the pier or abutment, the care and cost of sinking, the considerations of tilt and shift during sinking and the vertical and horizontal forces to which the well is subjected. A circular well has the minimum perimeter for a given dredge area and hence the ratio of sinking effort to skin friction is maximum. Also, since the perimeter is equidistant at all points from the centre of the dredge

hole, the sinking is more uniform than for other shapes. However, the disadvantage of a circular well is that in the direction parallel to the span of the bridge, the diameter of the well is much more than the minimum size required to accommodate the bridge pier and hence the circular well causes more obstruction to waterway

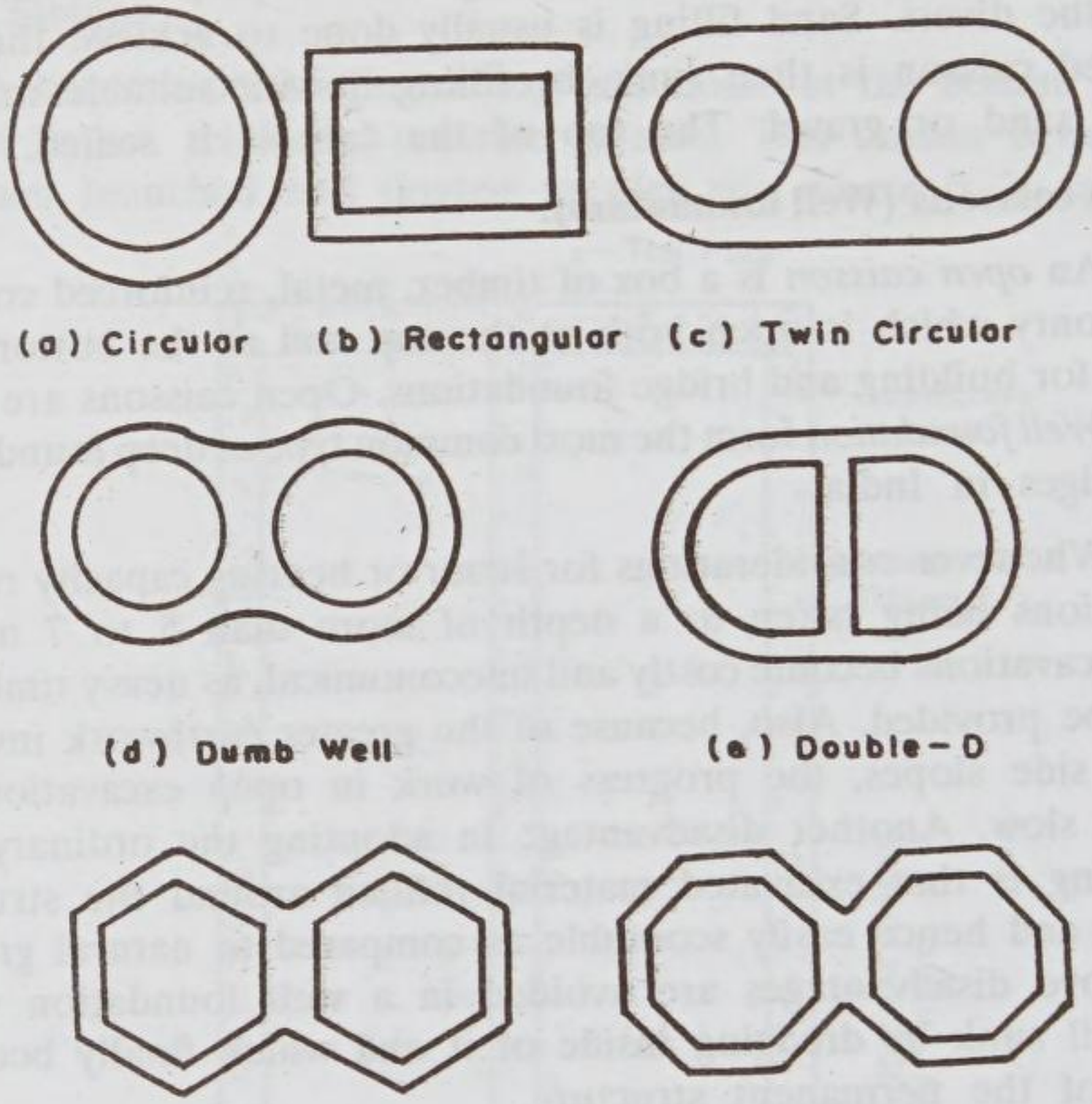


FIG. 4.34. SHAPES OF WELLS.

than the bridge pier does. This disadvantage is avoided in the case of a double-D shape which conforms to the shape of the bridge pier in plan. The dredge area is smaller for double-D. Hence for large piers, a double-D is more economical than a single circular well. Twin circular well aim at combining the advantage of a circular well and of a double-D, but the only snag is that the two wells sunk close to each other have a tendency to close in or move apart. However, in abutments and wing walls where the tilt and shift in position are not important, a battery of small diameter wells.

Fig. 4.35 shows a typical section of a well foundation with its component parts. The following components of a well have to be considered in the design of a well foundation :

(*i*) Well curb and cutting edge

(*ii*) Steining
(*iii*) Bottom plug
(*iv*) Well cap.

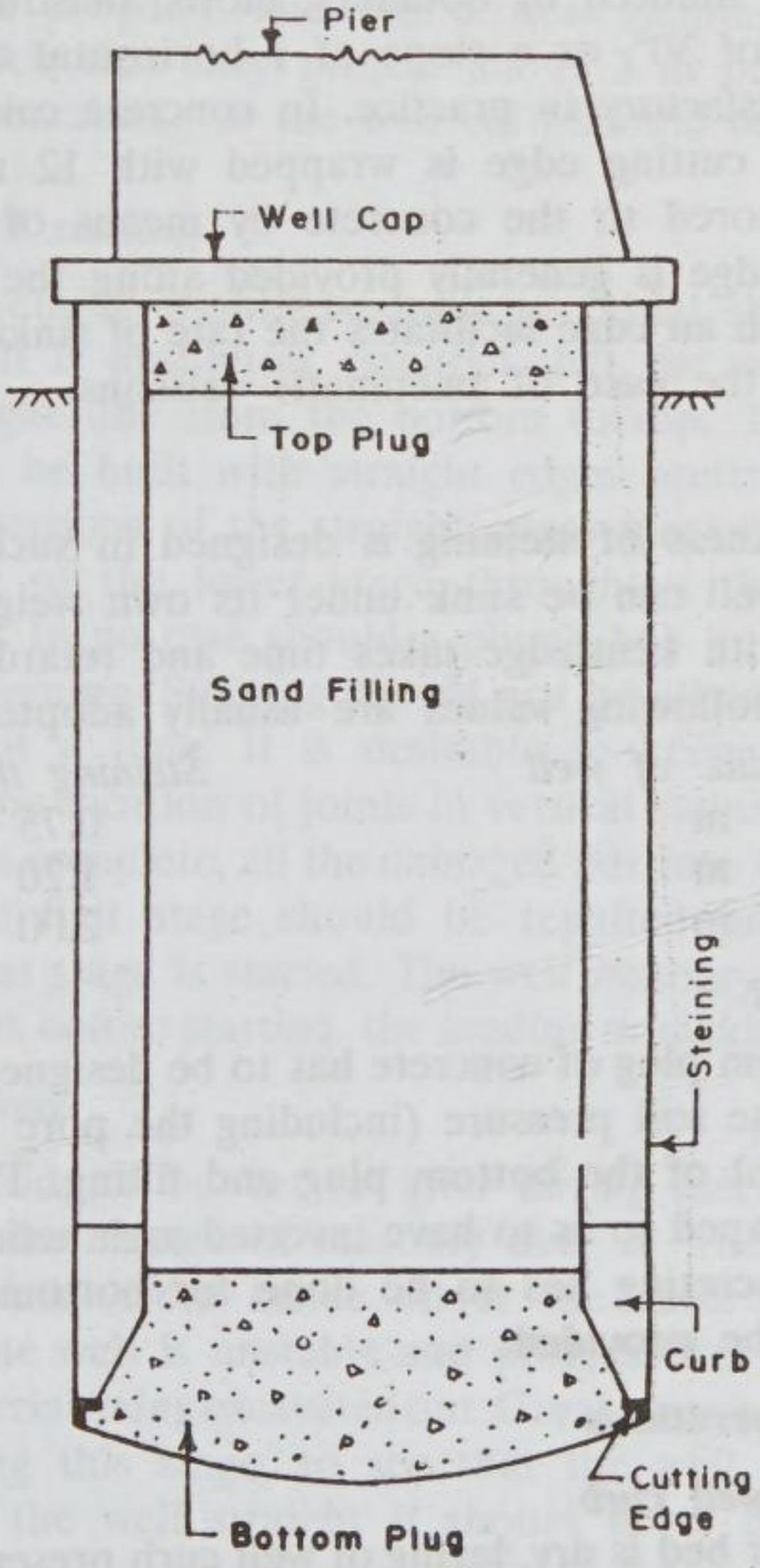


FIG. 4.35. SECTION OF A WELL FOUNDATION.

(i) Well curb.

The well curb is designed for supporting the weight of the well with partial support at the bottom of the cutting edge, i.e., when only part of the cutting edge is in contact with soil and the remaining portion is only held by skin friction. A three point support of the cutting edge resting on a log may be assumed for design purposes. The load coming on the well curb is uncertain as considerable part of it is borne by skin friction. The well curb has also to withstand stresses due to *sand blows* as well as due to light blasting required when boulder obstruct the sinking of the well.

(ii) Cutting edge

The cutting edge should have as sharp an angle as practicable for knifing into the soil without making it too weak to resist the various stresses induced by boulders, blows, blasting ete. An angle to the vertical of 30°, or a slope of 1 horizontal to 2 vertical has been found satisfactory in practice. In concrete caissons, the lower portion of the cutting edge is wrapped with 12 mm steel plates which are anchored to the concrete by means of steel straps. A sharp vertical edge is generally provided along the outside face of the caisson. Such an edge facilitates the rate of sinking and prevents air leakage in the case of pneumatic caissons.

(iii) Steining

The thickness of steining is designed in such a way that at all stages the well can be sunk under its own weight, as the need for weighting with kentledge takes time and retards progress considerably. The following values are usually adopted

Outside dia. of well	*Steining thickness*
3 m	0.75 m
5 m	1.20 m
7 m	2.00 m

(iv) Bottom plug

The bottom plug of concrete has to be designed for an upward load equal to the soil pressure (including the pore water pressure) minus self weight of the bottom plug and filling. The bottom plug is made bowl-shaped so as to have inverted arch action. As generally under-water concreting has to be done for bottom plug, no reinforcement can be provided.

Well sinking operations

1. *Laying the well curb*

If the river bed is dry, laying of well curb presents no difficulty. In such a case, excavation upto half a metre above subsoil water level is carried out and the well curb is laid. If, however, there is water in the river, suitable coffer dams are constructed around the site of the well and islands are made. The sizes of the island should be such as to allow free working space necessary to operate tools and plane for movement of labour etc. When the island is made, the centre point of the well is accurately marked and the cutting edge is placed in a level plane. It is desirable to insert wooden sleepers below the cutting edge at regular intervals so as to distribute the load and avoid setting of the cutting edge unevenly during concreting. These sleepers are, however, removed once the shuttering of the well curb has been stripped off. The inside shuttering of the curb

is generally made of brick masonry built to proper profile and plastered. The outer shuttering is made of wood or steel. Steel lined timber shuttering is preferable. All reinforcement of the curb should be placed in position properly, and the vertical steining bars should also be placed such that they project about 2 m beyond the top of the curb. All concreting in the well curb should be done in one continuous operation.

2. *Masonry in well steining*

The well steining should be built in initial short height of about 2 m only. It is absolutely essential that the well steining is built in one straight line from the bottom to top. To ensure this the steining must be built with straight edges preferably of angle iron. The lower portions of the straight edges must be kept butted with the masonry of the lower stage throughout the building of the fresh masonry. In no case should a plumb bob be used to build masonry in well steining. Steining should not be allowed to be built more than 5 m at a time. It is desirable to keep the stages of masonry work at the location of joints in vertical steining bars. After sinking one stage is complete, all the damaged portions of the steining at the top of the first stage should be repaired properly before masonry in the next stage is started. The well masonry is fully cured for at least 48 hours before starting the loading or sinking operations.

3. Sinking Operations

A well is ready to be 'set in' after having cast the curb and having built first short stage of masonry over it. The well is sunk by excavating material from inside under the curb. In the initial stage of sinking, the well is unstable and progress can be very rapid with only little material being excavated out. Great care should therefore be exercised during this stage, to see that the well sinks to true position. To sink the well straight it should never be allowed to go out of plumb.

Excavation and scooping out of the soil inside the well can be done by sending down workers inside the well till such a stage that the depth of water inside becomes about 1 m. After this stage, Jhams, worked by manual or animal power or by means of diesel, electric or steam whinches are used for excavating the material from inside and under water. When clay strata is to be pierced through, a rail chisel may be used. In case the soil is not very hard, but hard enough as not be excavaged by jhams, the use of phawrah jhams is effective. When power whinches are available, clayey strata can also be successively excavated with the help of big grabs having tempered steel teeth.

As the well sinks deeper, the skin friction on the sides progressively increases. To overcome the increased skin friction and the loss in weight of the well due to buoyancy, additional loading known as *kentledge* is applied on the well.

Pumping out the water from inside the well is effective in sinking of well under certain conditions. Pumping should be discouraged in the initial stage. Unless the well has gone deep enough or has passed through a ring of clayey strata so that chances of tilts and shifts are minimised during this process. Complete dewatering should not be allowed when the well has been sunk to about 10 m depth. Sinking thereafter should be done by grabbing, chiselling, applying kentledge and using gelignite charges. Only when these methods have failed, dewatering may be allowed upto depressed water level of 5m and *not more*.

On certain occassions, a well is struck up and normal method of kentledge and dredging fail to sink it further. In such a case, frictional resistance developed on its outer periphery is reduced considerably by forcing jet of water on the outer face of the well around. This method is effective in case the well is being sunk in sand strata.

4. *Tilts and shifts*

The primary aim in well sinking is to sink them straight and at the correct position. Suitable precautions should be taken to avoid tilts and shifts. Also, proper records of tilts and shifts should be maintained and measures should be taken to counteract tilts and shifts. The precautions to avoid tilts and shifts are as follows:

(*i*) The outer surface of the well curb and steinings should be as regular and smooth as possible.

(*ii*) The radius of the curb should be kept 2 to 4 cm larger than the outside radius of well steining.

(*iii*) The cutting edge of the curb should be of uniform thickness and sharpness since the sharper edge has a greater tendency of sinking than a blunt edge.

(*iv*) The dredging should be done uniformly on all sides in circular well and in both pockets in a twin well. The tilts and shifts of well, if any, must be carefully checked and recorded. The correct measurement of the tilt at any stage is perhaps one of the most important field operations during well sinking.

(*v*) As soon as tilt exceeds 1 in 200, the sinking should be supervised with special care and rectifying measures should be immediately taken.

5. *Completion of well*

When the well bottom has reached the desired strata, further sinking of the well is stopped. A concrete seal(plug) is provided at the bottom. The bottom plug is made bowl-shaped so as to have inverted arch action. As generally under watering concreting has to be done, no reinforcement can be provided. Under water concreting is done with the help of tremie. However, if it is possible to dewater the well successfully, concrete can be placed in dry also.

After having plugged the well at its bottom, the interior space of the well is filled either with water or sand. It may even be kept empty. The well is capped at its top, with the help of reinforced concrete slab. If however, sand has been filled inside, top plug of lean concrete is interposed between the well cap and sand filling, as shown in Fig. 4.36.

3. Pneumatic Caissons

Pneumatic caissons are closed at the top and open (during construction) at the bottom. The essential feature of a pneumatic Caissons is that compressed air is used to exclude or remove water from the working chamber at the bottom, and the excavations are thus carried out in dry conditions. The method of construction of pneumatic caisson is similar to that for open caissons (wells) except that the working chamber is kept air tight. In order that the subsoil water may not enter the working chamber, the pressure of air in the shaft is kept just higher than that of water at that depth. However, the maximum pressure is limited from the considerations of health of persons who work inside the chamber. Normally, the tolerable air pressure under which a man can work is limited 3.5 kg/cm^2

Let h be the height of water, at any stage of working. Then air pressure p required to exclude water is given by

$$p = \frac{w\,h}{10000}$$

where p = air pressure, in kg/cm^2

w = unit weight of water

= 1000 kg/m^3

h = head of water in metres.

$$\therefore \quad p = \frac{1000\,h}{10000} = 0.1\,h \qquad \ldots(4)$$

If $h = 1$ m , $p = 0.1$ kg/cm^2

If maximum air pressure is limited to 3.5 kg/cm^2 , the limiting head of water is given by

$$h_{lim} = \frac{3.5}{0.1} = 35 \text{ m.}$$

This is the maximum value. However, pneumatic Caissons are adopted *only* if the head of water is more than 12 m. Thus a pneumatic Caissons can be used for depths of water ranging from 12 to 35 m.

Sinking of pneumatic Caissons is tedious, time consuming and expensive. However, theser are adopted at places where it is difficult to use bulky equipment required for sinking wells. Another advantage of pneumatic caissons is that the entire process of sinking of well is carried out under controlled conditions. It affords easy inspection of work.

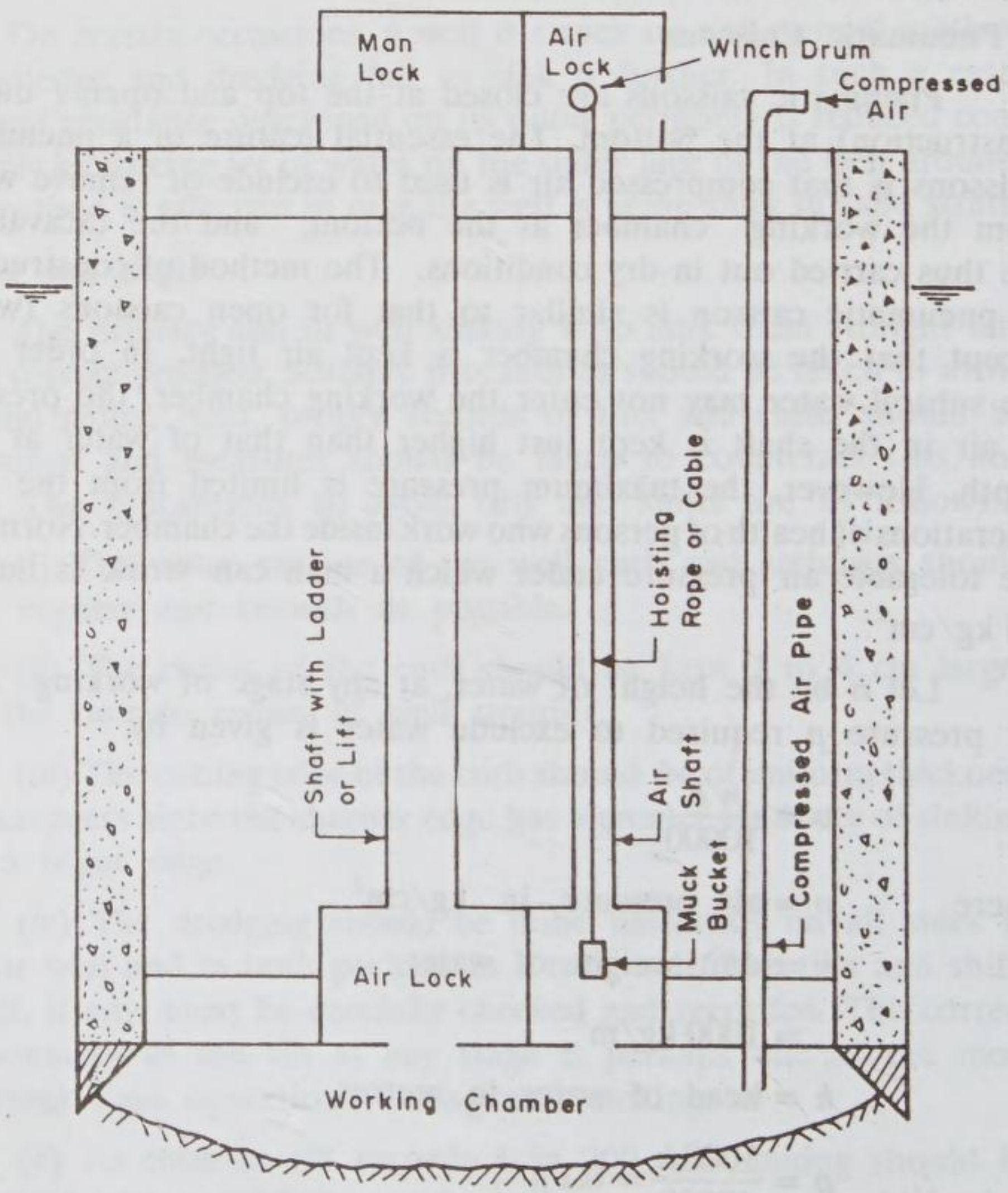


FIG. 4.36. SECTIONS OF A PNEUMATIC CAISSON.

Fig. 4.36 shows a typical section of a pneumatic Caisson. The

procedure for sinking the pneumatic well is as follows :

1. The caisson is sunk exactly in the same manner, as used for well sinking till the depth of water is shallow, and no trouble is encountered in sinking the well.

2. When the presence of water poses problems, an air-lock is placed inside the well. The air lock may rest on rubber seals, just above the cutting edge. The number of air locks may vary from one to three. Generally, two air locks are used–one for sending men inside and the other for removing the excavated material with the help of a muck bucket and hoisting rope.

3. After properly placing the air lock in position, so that direct air entry is sealed, water is pumped out from the bottom, and air pressure is gradually increased so that fresh water does not enter the working chamber.

4. Labourers are then sent down to the working chamber through the appropriate air lock. In order to prevent leakage of air, arrangement of double gates is provided. The person enters the first gate, where the pressure is atmospheric. The first door is closed and pressure is gradually increased to make it equal to the one in the working chamber. The water then enters the working chamber through the second door which is immediately closed. The reverse process is adopted for bringing the person out of working chamber. The height of working chamber is kept about 2 m, with proper lighting arrangement. Air is supplied through the inlet pipe connected to an air compressor.

5. Excavation is carried out in the working chamber by the labourers sent down through air lock. The excavated material is sent up through the muck bucket lifted up by a hoisting rope operated by whinch drum, through the air lock. In order to assist sinking, air pressure may be reduced for a short while. Sometimes, explosives may be employed, in which case it is essential to make arrangements for the immediate removal of the foul fumes.

6. When the caisson bottom has reached the desired level, concrete seal (or plug) is made by concreting upto the underside roof of the working chamber. Sufficient air pressure is maintained to force the concrete against the bottom surface, till it hardens.

7. Air locks are removed, well is filled with sand or water (or even kept empty). Well cap is then formed out its top as usual.

Advantages of pneumatic Caissons

1. Work is done in dry condition, thus imparting better control over the work. Also, foundation preparations are better.

2. Verticality of pneumatic Caisson is easier to check and control, as compared with open caissons(wells).

3. Since concrete is placed in dry, good and reliable quality work is obtained.

4. Obstruction from boulders or logs can be easily removed. Excavation by blasting may be easily done, if found necessary.

Disadvantages

1. Cost of construction is high.
2. High degree of skill is required in sinking.
3. Proper health controls are necessary for the labourers.
4. Depth of penetration below water level is limited to 35 m.

PROBLEMS

1. Explain the situations in which the pile foundation is preferred.
2. Classify various types of piles based on (*i*) function and (*ii*) materials and composition.
3. Differentiate clearly between (*i*) cased cast-in-situ pile, (*ii*) uncased cast-in-situ pile, and (*iii*) bored pile. Give one example of each.
4. Explain, with the help of sketches, the method of formation of Raymond standard pile and step-taper-pile.
5. What is the difference between Mac Arthur cased pile and Mac Arthur padestal pile ? Explain with the help of sketches.
6. Write notes on the following:

   (*a*) Swage pile

   (*b*) Western button-bottom pile.

7. Explain, with the help of sketches, the method of forming simplex pile. What is alligator jaw point ?
8. Draw typical sketches showing stages in the formation of Franki pile.
9. What is the difference between standard vibro-pile and vibro- expanded pile. Draw neat sketches for the finished piles.
10. Explain the method of forming cast-in-situ pressure piles.
11. (*a*) Draw a typical sketch of octagonal pre-cast pile.

    (*b*) Write advantages and disadvantages of precast concrete piles.
12. Write a note on use of H-piles.

13. Sketch a typical timber pile. Write advantages and disadvantages of timber piles.

14. Explain, with the help of sketches, the method of forming timber concrete composite pile. When do you use such a pile.

15. Write short notes on : (*a*) Screw pile

(*b*) Disc pile

(*c*) Sheet pile.

16. (*a*) Draw a neat sketch showing a drive cap or helmet, along with the cushion.

(*b*) Explain in brief various types of drive hammers used for pile driving.

17. What do you understand by dynamic formulae. Write Engineering News Formula for estimating the load carrying capacity of a pile.

18. Explain Hiley's formula, adopted by the Indian Standard, for the estimation of ultimate load of a pile.

19. Differentiate between static formulae and dynamic formulae used for determining the load-carrying capacity of piles. Comment on the use oi dynamic formulae.

20. A concrete pile is being driven with a drop hammer weighing 2500 kg (25 kN) and having a free fall of 1.2 m. The penetration in the last blow is 4 mm. Using Engineering News Formula, determine the load carrying capacity of the pile.

21. A reinforced concrete pile weighing 4 tonnes (40 kN), inclusive of helmet and dolly, is driven by a drop hammer weighing 3.5 t (35 kN) and having an effective fall of 1 m. The average set per blow is 1.2 cm. The total temporary elastic compression is 1.6 cm. Assuming the co-efficient of restitution as 0.25 and a factor of safety of 2, determine ultimate bearing capacity and allowable load for the pile.

22. Describe pile load test for determining the bearing capacity of a pile.

23. Write a note on design of pile cap.

24. Explain how do you determine the efficiency of pile group, using Feld's rule.

25. What do you understand by under-reamed pile foundation ? Where do you use it ? Draw a typical sketch of under-reamed piles foundation, along with the grade beam, for use in expansive soil.

26. Explain, with the help of sketches, the method of forming under-reamed piles foundation.

27. Differentiate between under-reamed pile foundation and bored compaction pile. Explain the method of forming bored compaction pile.

28. Write notes on : (*i*) Sand pile

(*ii*) Concrete sheet pile

(*iii*) Wooden sheet pile.

29. What is a coffer dam ? Where do you use it ? With the help of sketches, explain in brief various types of coffer dams.

30. What is a box caisson ? When do you use it ? Explain the method of installing a box caisson.

31. Explain, with the help of a sketch, the components of a well foundation. How do you construct a well curb ?

32. Write a note on well sinking operations.

33. With the help of a neat sketch, explain the method of sinking a pneumatic Caisson. What is the optimum depth under water upto which you can sink a pneumatic Caisson ?

5

Masonry-1 : Stone Masonry

5.1. MASONRY

Masonry may be defined as the construction of building units bonded together with mortar. The building units (commonly known as *masonry units*) may be stones, bricks or precast blocks of concrete. When stones are used as the building units or building blocks, we have *stone masonry.* Similarly, in *brick masonry*, bricks are used as the building units. A composite masonry is a construction in more than one type of building units. *Masonry work* is one of the major building crafts and one of the oldest. It has built itself great reputation as one of the premier traditional materials of building. Even though new principles of construction and new materials become prominent in building construction practices, masonry has got the highest importance in building industry.

Masonry is normally used for the construction of foundations, walls, columns and other similar structural components of buildings. The basic advantage of masonry construction lies in the fact that in load-bearing structures, it performs a variety of functions such as (*i*) supporting loads, (*ii*) subdividing space, (*iii*) providing thermal and acoustic insulation, (*iv*) affording fire and weather protection etc., which in a framed structure has to be provided separately. Earlier, the use of masonry construction had its limitations in multistoreyed buildings. The 16 storey 'Monadnock Building' in Chicago designed by John Rort (1891) has 180 cm thick brick walls at the base. However, extensive research, including large scale testing, has been carried out in regard to the behaviour of masonry which has enabled engineers to design tall masonry structures on sound engineering principles with greater exactitude, economy and confidence. There are recent examples of masonry construction in advanced countries in which 12 to 20 storey *load–bearing masonry buildings* have only 25 to 40

cm thick walls.

Depending upon the type of building units used, masonry may be of the following types :

1. Stone masonry
2. Brick masonry
3. Hollow concrete blocks masonry
4. Reinforced brick masonry
5. Composite masonry.

In this chapter, stone masonry has been discussed. Other types of masonry have been discussed in subsequent chapters.

5.2. DEFINITION OF TERMS USED IN MASONRY

Following are some of the technical terms used in masonry work. Since these terms are frequently used in the description and procedures, it is essential to understand the meaning of these terms. The terms which apply exclusively to the brick masonry have been defined separately in the next chapter.

1. Course. A course is a horizontal layer of masonry unit. Thus, in stone masonry, the thickness of a course will be equal to the height of the stones plus thickness of one mortar joint. Similarly, in brick masonry, the thickness of a course will be equal to the thickness of modular brick plus thickness of one mortar joint.

2. Header. A header is a full stone unit or brick which is so laid that its length is perpendicular to the face of the wall. Thus, the longest length of a header lies at right angles to the face of the work. In the case of stone masonry header is sometimes known as *through stone.* In the case of modular bricks, a brick header will show its face measuring 10 cm × 20 cm on the face of the wall.

3. Stretcher. A stretcher is a full stone unit or brick which is so laid that its length is along or parallel to the face of the wall. Thus, the longest length of stretcher lies parallel to the face of the work. Thus, in the case of modular bricks, a brick stretcher will show its face measuring 10 cm × 20 cm.

4. Header Course. A course of brick work showing only headers on the exposed face of the wall is known as header course or heading course. Thus a header course of bricks will show all the brick units measuring 10 cm × 10 cm of the face of the wall.

5. Stretcher Course. A course of brick work showing only the stretchers on the exposed face of the wall is known as the strecher course or stretching course.

6. Bed. This is the lower surface of a brick or stone in each course. This is the surface of stone or brick perpendicular to the line of pressure.

7. Natural bed. Building stones are obtained from rocks which have distinct planes of divisions along which the stones can be easily split. This plane is known as natural bed. In stone masonry, the direction of natural bed should be perpendicular to the line of pressure.

8. Bond. Bond is a term in masonry, applied to the over-lapping of bricks or stones in alternate courses, so that no continuous vertical joints are formed and the individual units are tied together.

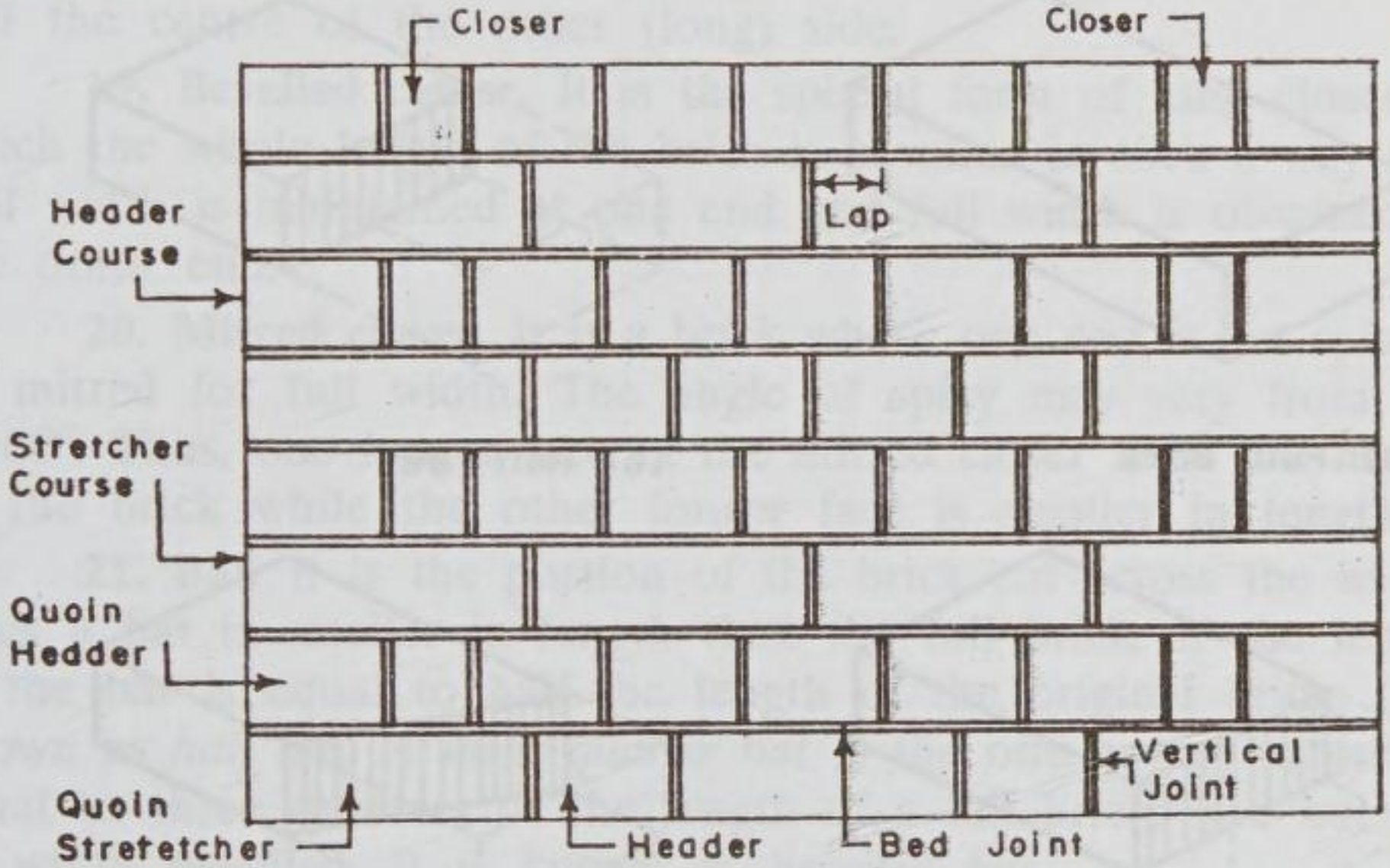


FIG. 5.1. ILLUSTRATION OF VARIOUS TERMS.

9. Quoins. The exterior angle or corner of a wall is known as quoin. The stones or bricks forming the quoins are known as stone quoins or quoin bricks. If the quoin is laid in such a manner that its width is parallel to the face of the wall, it is known as quoin header. If, however, the length of the quoin is laid parallel to the face of the wall, it is known as quoin stretcher. Quoin stones are selected sound and large and their beds are properly dressed.

10. Face. It is the surface of the wall exposed to the weather.

11. Back. The inner surface of the wall which is not exposed to weather is termed as *back.*

12. Facing. The material used in the face of the wall is known as facing.

13. Hearting. The inner portion of the wall between the facing and backing is known as the hearting.

14. Side. It is the surface forming the boundary of bricks or stones in a direction transverse to the face and bed.

15. Joint. The junction of adjacent units of bricks or stones is known as a joint. Joints parallel to the bed of bricks or stones is known as *bed joint.* Bed joints are thus horizontal mortar joints

upon which masonry courses are laid. Joints perpendicular to the face of the wall is known as *cross-joint or vertical joints*. All joints are formed in cement mortar, lime mortar or mud mortar. A joint which is parallel to the face of the wall is known as wall joint.

16. Closer. It is the portion of brick cut in such a manner that its one long face remains uncut. Thus, a closer is a header of small width.

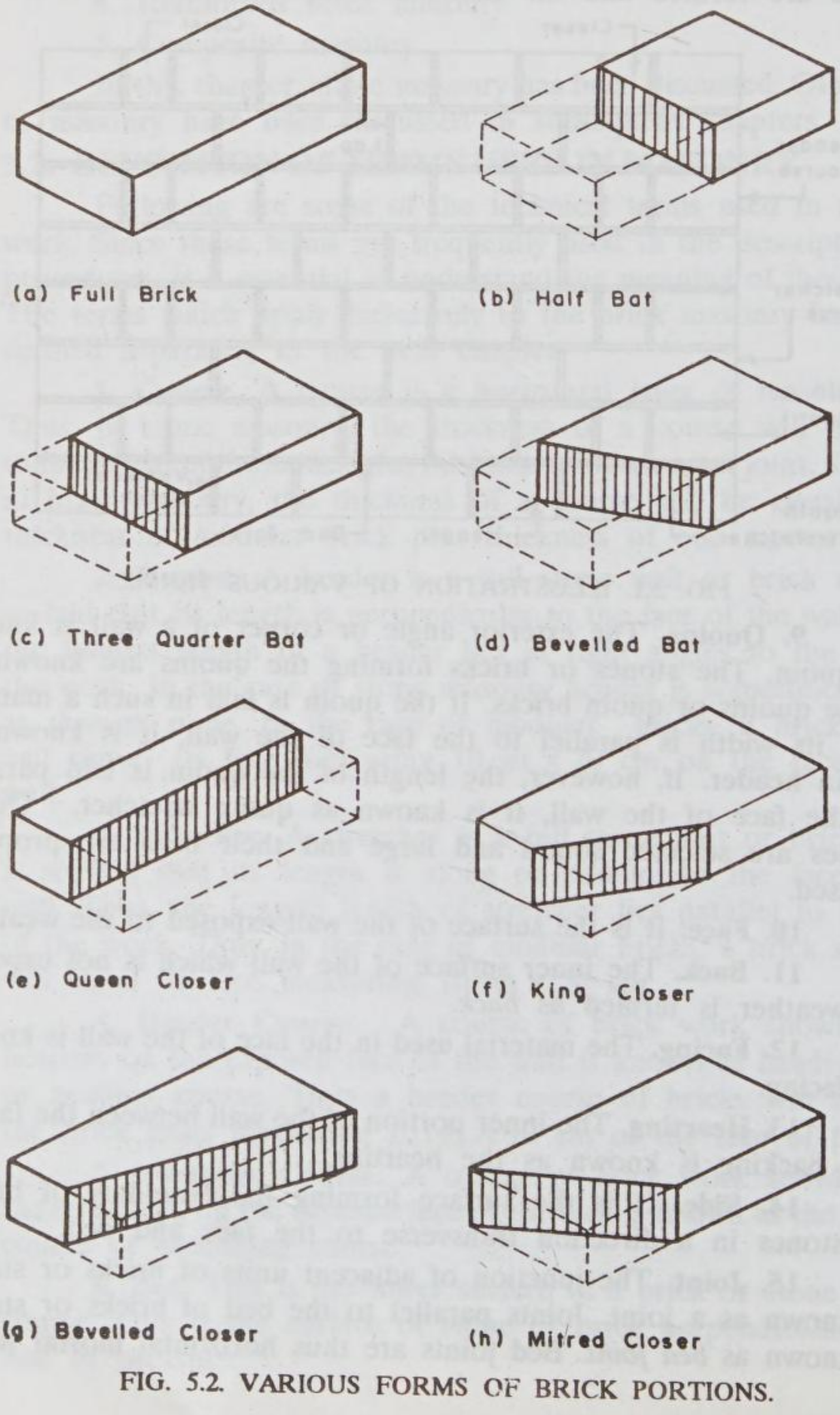


FIG. 5.2. VARIOUS FORMS OF BRICK PORTIONS.

17. Queen closer. It is the portion of a brick obtained by cutting a brick length-wise into two portions. Thus, a queen closer is a brick which is half as wide as the full brick.

18. King closer. It is the portion of a brick which is so cut that width of one of its end is half that of a full brick, while the width at the other end is equal to the full width. It is thus obtained by cutting off the triangular piece between the centre of one end and the centre of the other (long) side.

19. Bevelled closer. It is the special form of king closer in which the whole length of the brick is bevelled in such a way that half width is maintained at one end and full width is obtained at the other end.

20. Mitred closer. It is a brick whose one end is cut splayed or mitred for full width. The angle of splay may vary from 45° to 60°. Thus, one longer face of the mitred closer is of full length of the brick while the other longer face is smaller in length.

21. Bat. It is the portion of the brick cut across the width. Thus a bat is smaller in length than the full brick. If the length of the bat is equal to half the length of the original brick, it is known as *half bat. A three quarter bat* is the one having its length equal to three quarters of the length of a full brick. If a bat has its width bevelled, it is known as *bevelled bat.*

22. Perpend. It is that vertical joint on the face of the wall, which lies directly above the vertical joints in alternate courses.

23. Frog. It is an indentation or depression on the top face of a brick made with the object of forming a key for the mortar. This prevents the displacement of the brick above.

24. Through Stone. A through stone is a stone header. Through stones are placed across the wall at regular interval. If the thickness of the wall is small, through stone may be of length equal to the full width of the wall. If, however the wall is considerably thick, two through stones with an overlap are provided, as shown in Fig. 5.3 (*b*). Through stones should be strong, and non-porous, and should be of sufficient thickness.

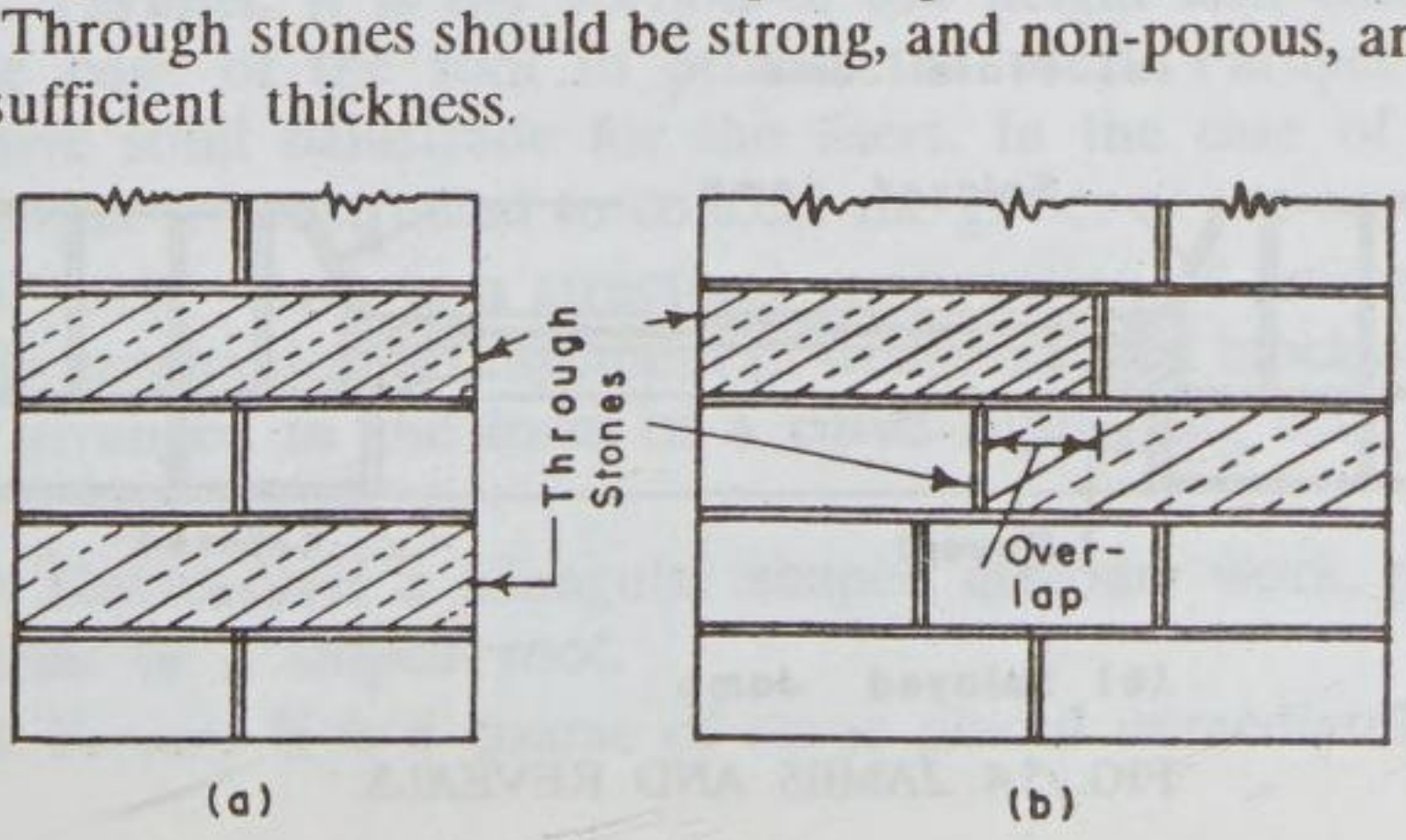


FIG. 5.3. THROUGH STONES.

25. Sill. The bottom surface of a door or a window opening is known as a sill. Sill is thus the horizontal member of brick, stone, concrete or wood provided to give support for the vertical members of the opening, and also to shed off rain water from the face of the wall immediately below the opening. Sill stones, when provided, are so dressed that they prevent the entry of water to the interior of the building.

26. Lintel. It is a horizontal member of stone, brick, wood, steel, or reinforced concrete, used to support the masonry and the super-imposed load above an opening.

27. Plinth. Plinth is the horizontal projecting course of stone or brick, provided at the base of the wall above the ground level. Plinth raises the level of ground floor above the natural ground level, thus protecting the building from rain, water, froast and other weather effects.

28. Plinth course. It is the uppermost course of the plinth masonry.

29. String course. It is the continuous horizontal course of masonry, projecting from the face of the wall for shedding rain water off the face. It is generally provided at every floor and sill level. A string course breaks the monotony of a plane surface, and thus imparts aesthetic appearance to the structure. The string course is suitably *weathered and throated* so as to throw off water clear of the wall surface.

30. Jambs. Jambs are the vertical sides of a finished opening

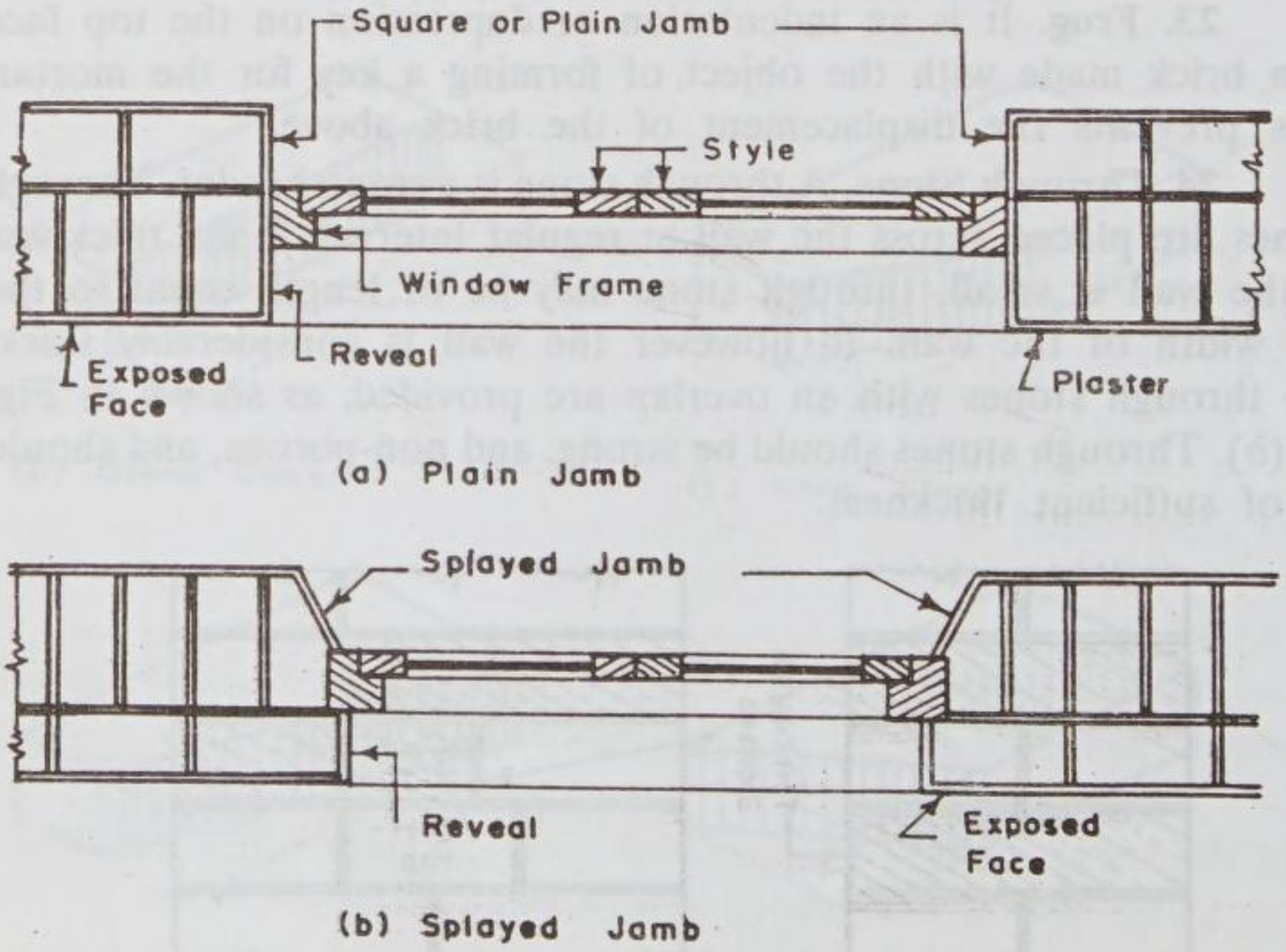


FIG. 5.4. JAMBS AND REVEALS.

for the door, window or fire place etc. Jambs may be plain or splayed or may be provided with the recess to receive the frames of doors and windows.

31. Reveals. These are the exposed vertical surfaces left on the sides of an opening after the door or window frame has been fitted in position.

32. Corbel. A corbel is a projecting stone which is usually provided to serve as support for joist, truss, weather shed etc. Corbels are generally moulded and given ornamental treatment. Corbels should extend atleast two-thirds of their length into the wall, so that they do not overturn or come out of the wall.

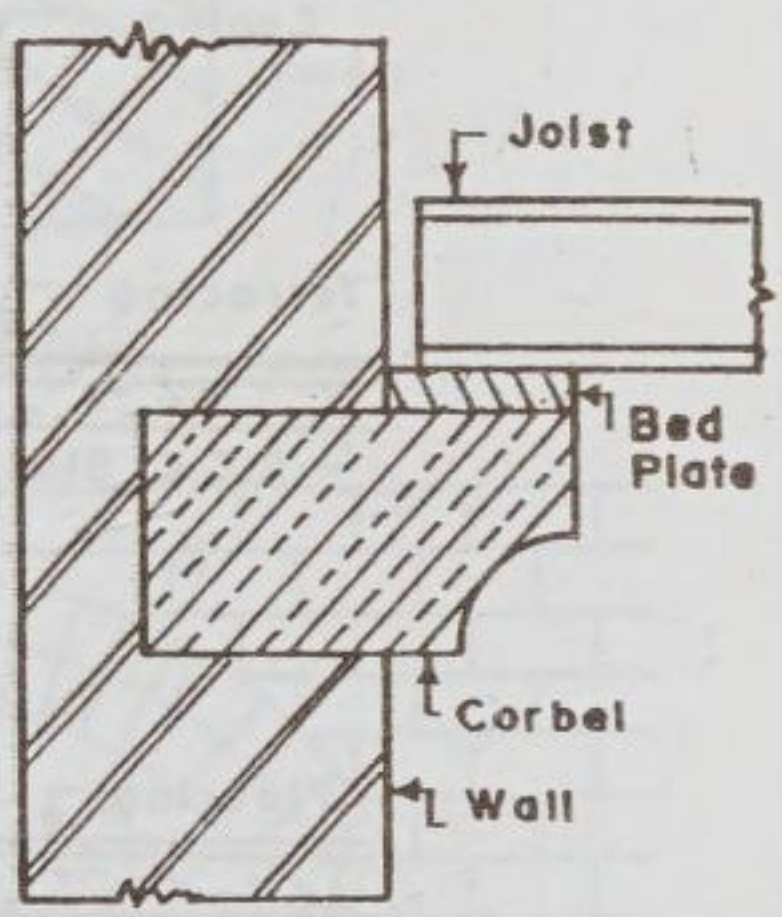


FIG. 5.5. CORBEL.

33. Cornice. It is a projecting ornamental course near the top of a wall or at the junction of wall and the ceiling. It penetrates the full width of the wall. It is weathered and throated to dispose off rain water. In order to prevent overturning of cornice, extra weight in the form of parapet wall should be provided (see Fig 5.6).

34. Coping. It is a covering of stone, concrete, brick of terracota, placed on the exposed top of a wall, to prevent seepage of water. It may also be provided on the top of compound wall. A coping is suitably weathered and throated (Fig. 5.7).

35. Weathering. It is the term used to denote the provision of the slope on the upper surface as sills, cornices, string courses, copings etc.

36. Throating. It is a groove provided on the underside of projecting elements such as sills cornices, copings etc., so that rain water can be discharged clear of the wall surface.

37. Parapet. It is the portion of low height wall constructed along the edge of the roof to protect the users. Parapet acts as a protective solid balustrade for the users. In the case of pitched roofs, parapet is constructed to conceal the gutter at the eaves level.

38. Arch. Arch is a structural construction of masonry constructed by mechanical arrangement of wedge–shaped blocks of stone or brick arranged in the form of a curve supporting wall or load above the opening.

39. Gable. It is a triangular shaped masonry work, provided at the ends of a sloped roof.

40. Freeze. It is a course of stone placed immediately below

the cornice, along the external face of the wall, intended to improve the appearance of the wall.

41. Blocking course. It is another course of stone placed immediately above the cornice. Apart from improving the appearance of the wall, it adds to the stability of the cornice against overturning.

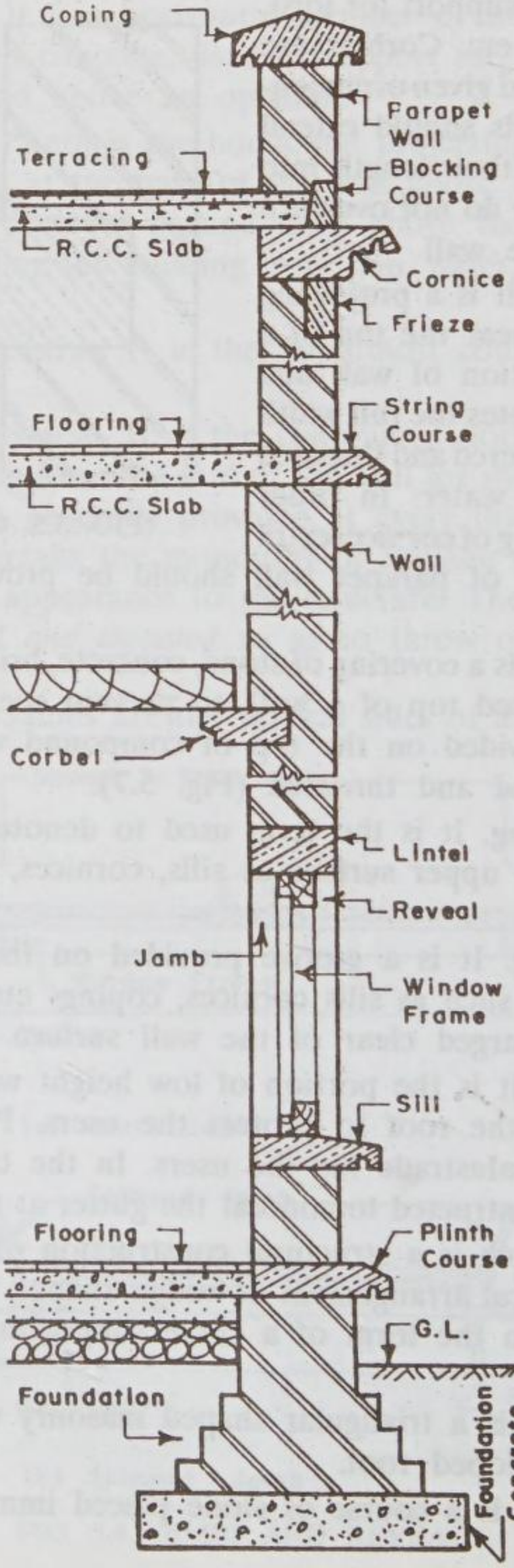


FIG. 5.6. SECTION THROUGH A WALL.

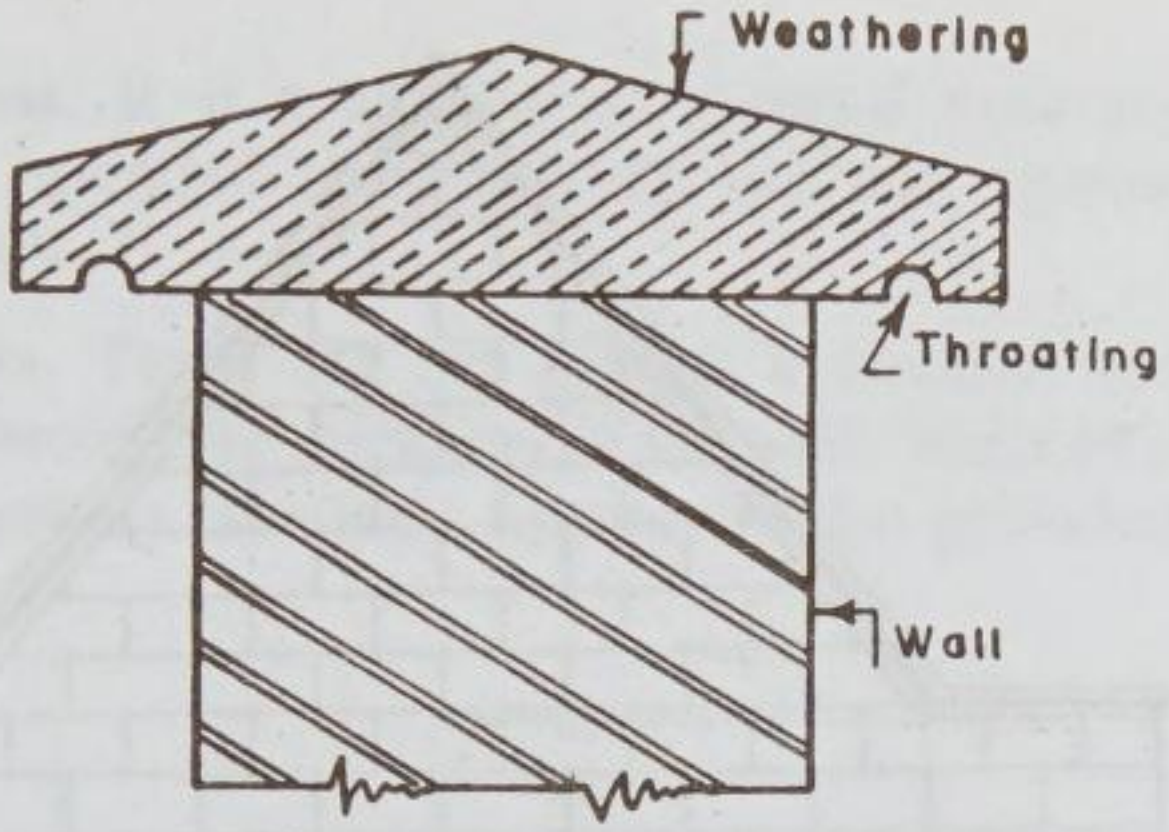


FIG. 5.7. COPING.

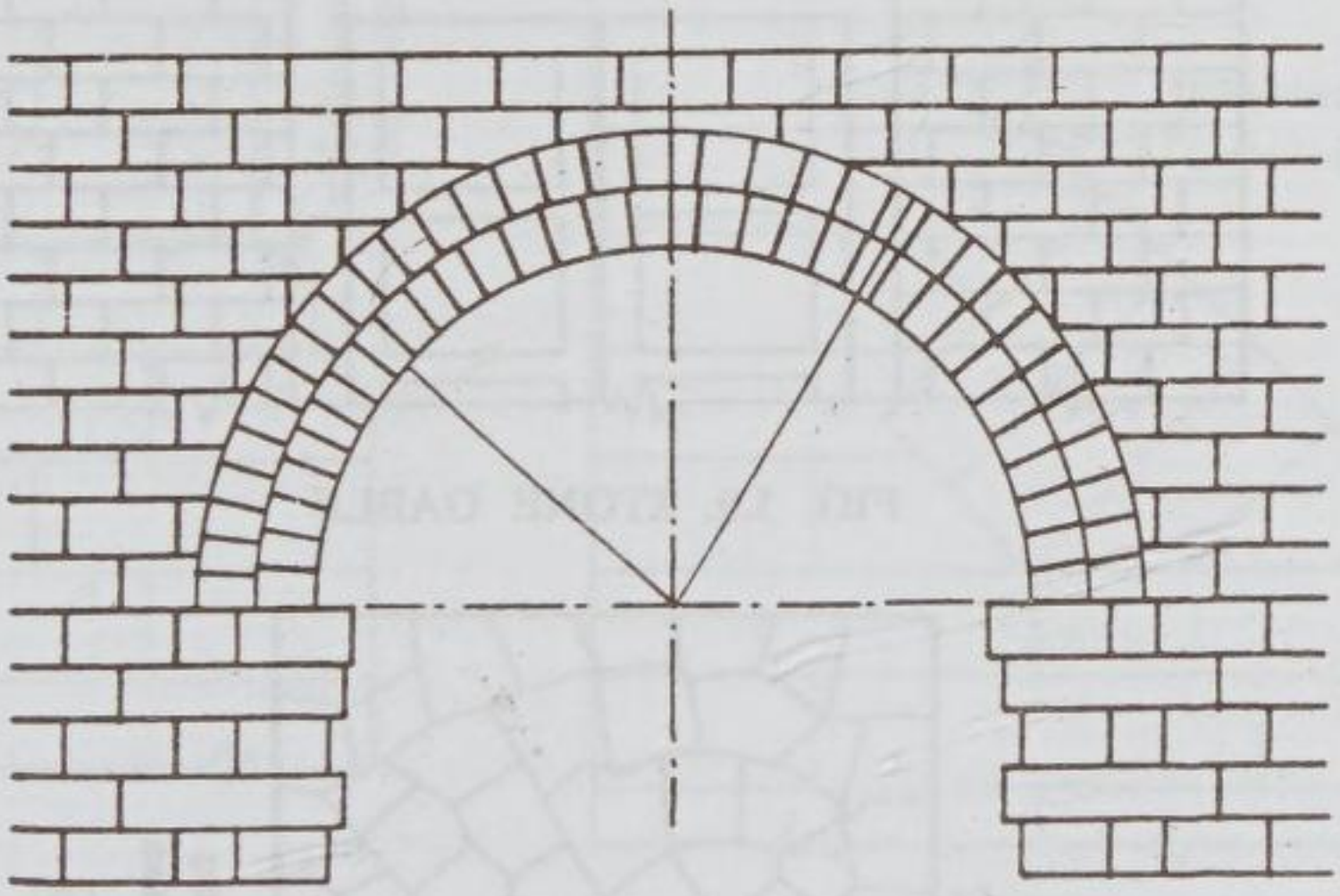


FIG. 5.8. ARCH.

42. Toothing. These are the bricks left projecting in alternate courses for the purposes of bonding future masonry work.

43. Lacing course. It is the horizontal course of stone blocks provided to strengthen a wall made of irregular courses of small stones, as shown in Fig. 5.10. The Lacing course may be either in ashlar masonry or coursed rubble masonry or brick masonry.

44. Spalls. Spalls are the chips or small pieces of stones obtained as a result of reducing big blocks of stones into the regular stone blocks. These spalls are used in filling the interstices of stone masonry.

45. Stoolings. These are the horizontal stones provided to receive jambs and mullions. These are formed at the ends of sills, transomes and heads.

46. Template or bed block. It is defined as the block of stone or concrete provided under a beam or girder to distribute the concentrated load over a greater area of the bearing surface.

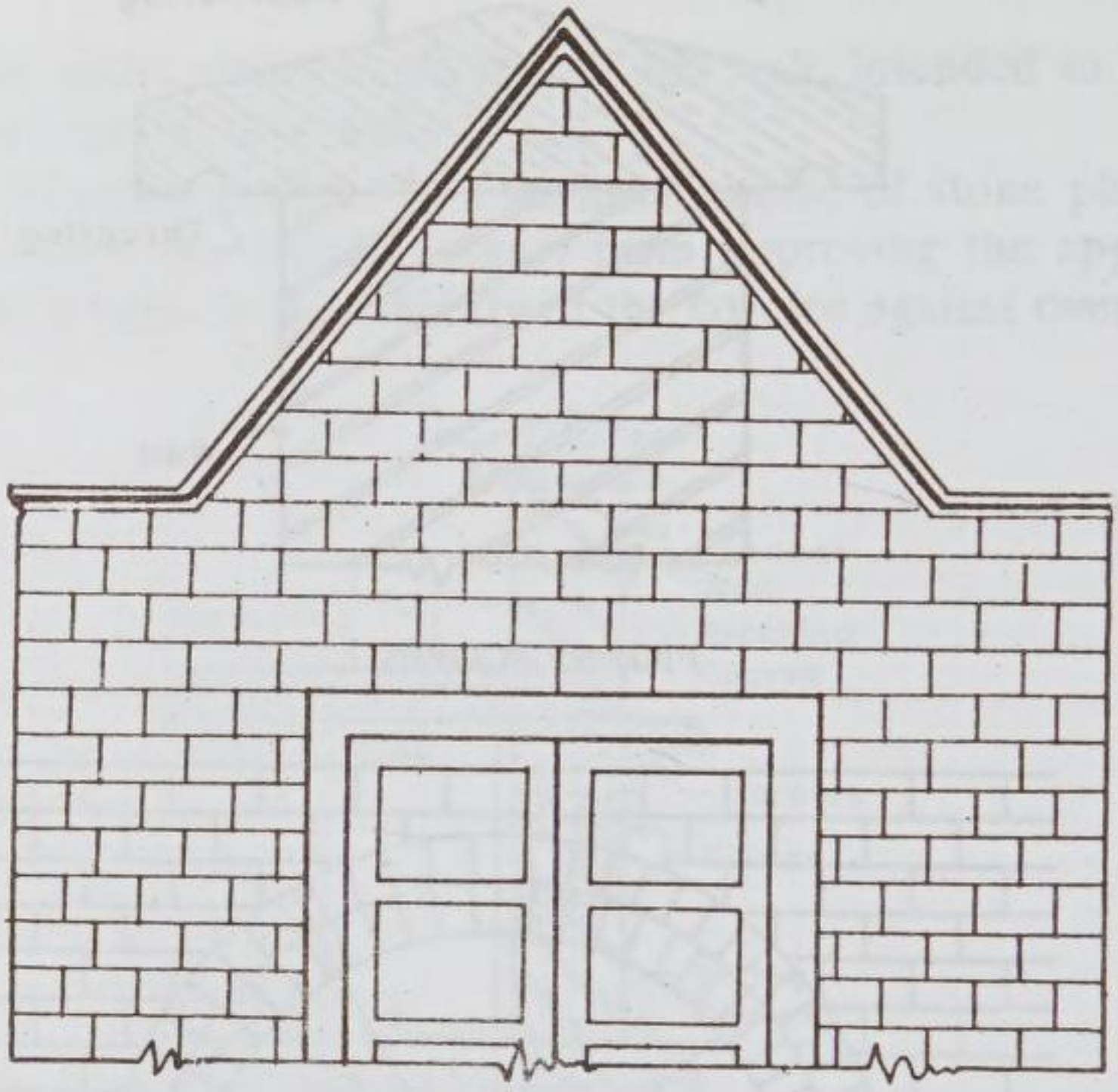


FIG. 5.9. STONE GABLE

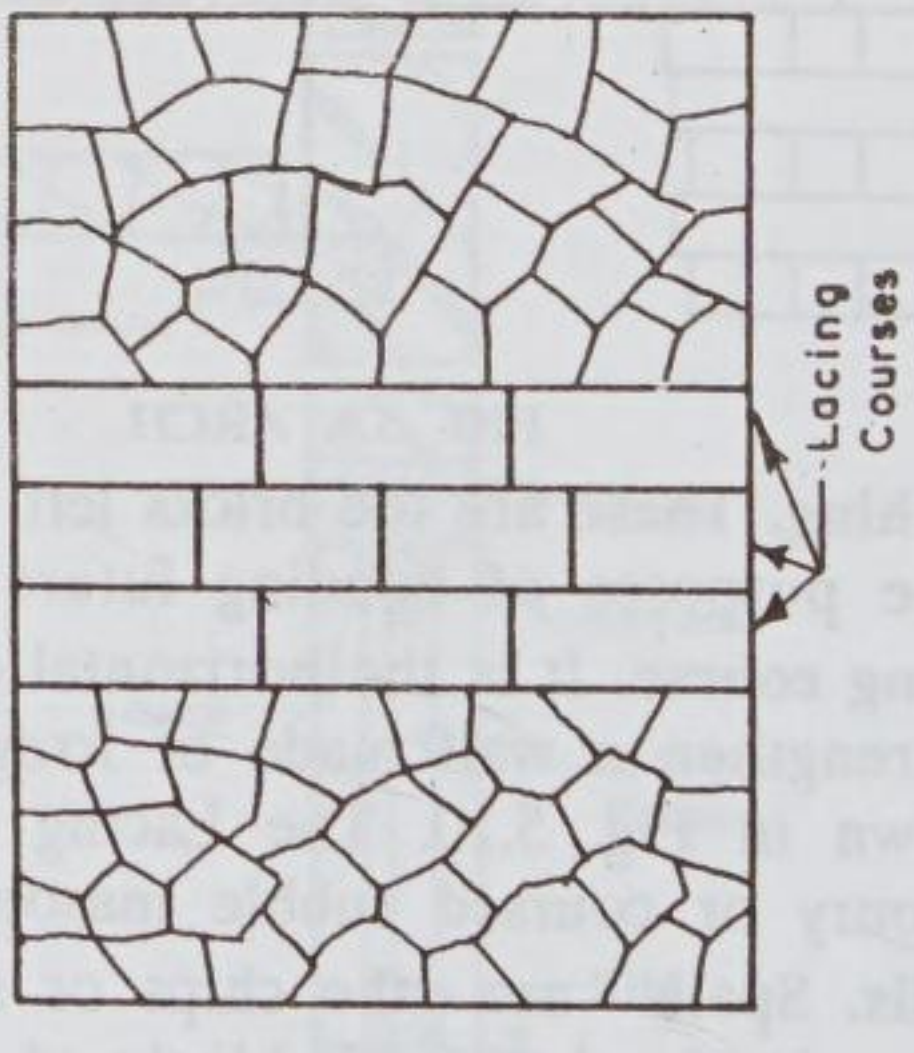


FIG. 5.10.

47. Column. It is a vertical load bearing member of masonry, which is constructed in an isolation from the wall, and whose width does not exceed four times its thickness.

48. Pier or Pilaster. Pier is an isolated vertical mass of stone or brick masonry to support beams, lintels, arch etc, the width of which exceeds four times its thickness. If it is made monolithic with the wall and projecting a little beyond to support the ends of a beam or truss etc, then it is called a pilaster.

49. Buttress. It is a sloping or stepped masonry projection from a tall wall intended to strengthen the wall against the thrust of a roof or arch.

50 Offsets. These are the narrow horizontal surfaces which are formed by reducing the thickness of the wall. Walls of tall buildings are formed with offsets. Similarly, offsets are also provided in masonry footings.

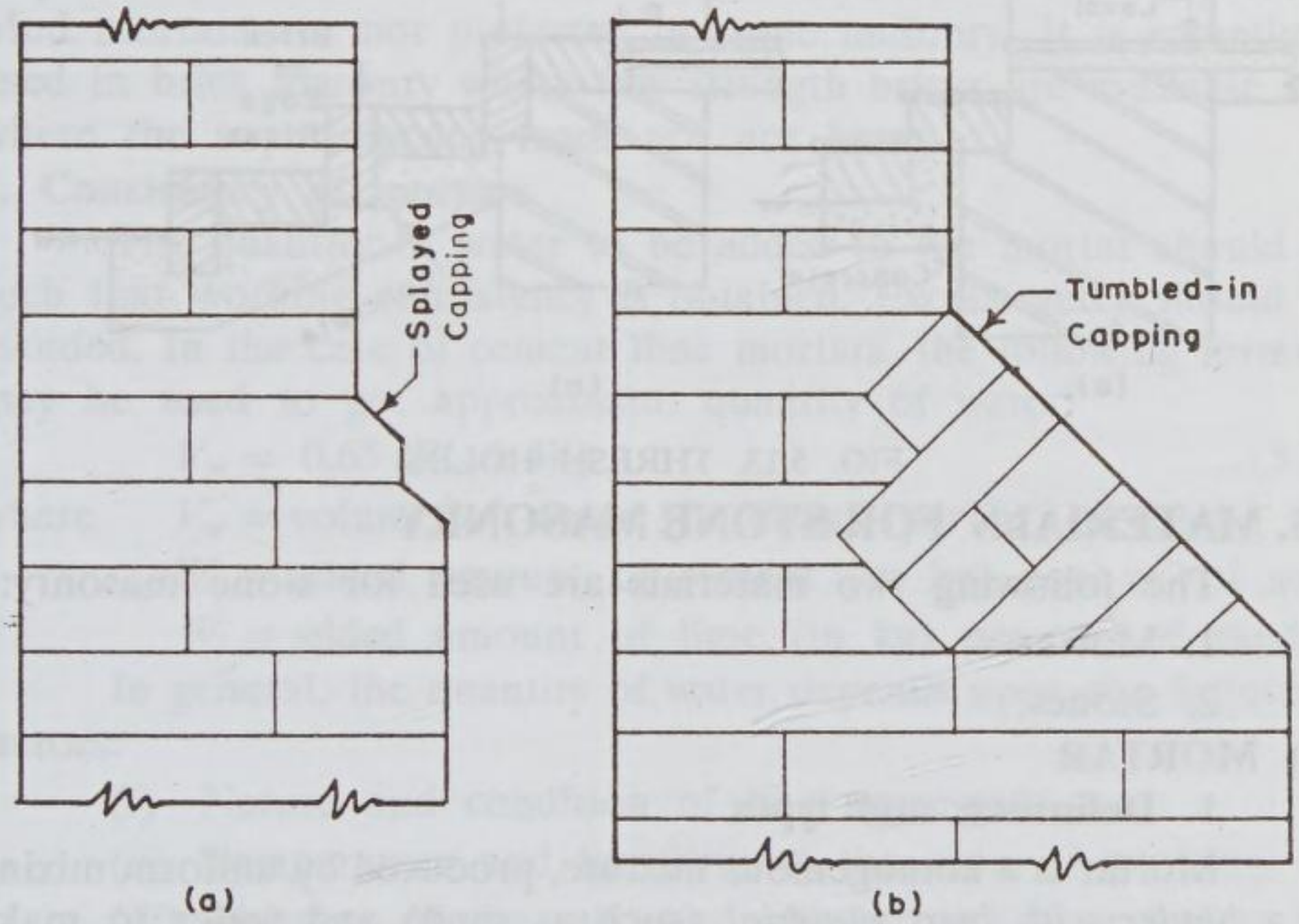


FIG. 5.11. BUTTRESSES.

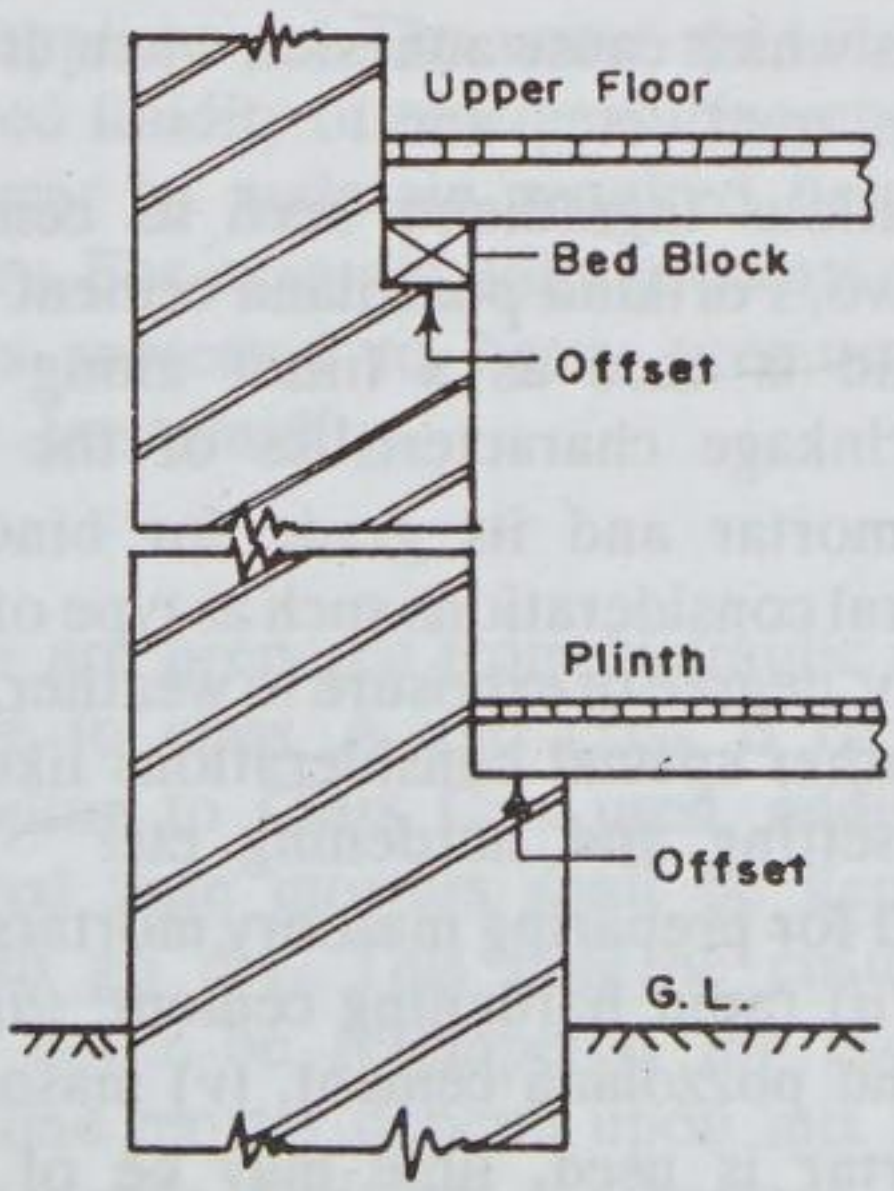


FIG. 5.12. OFFSETS.

51. Thresh holds. Thresh hold is the arrangement of steps provided from the plinth level of external door or verandah to the ground level. These may consist of stone, brick or concrete, and are constructed at the last stage of construction activities of the building.

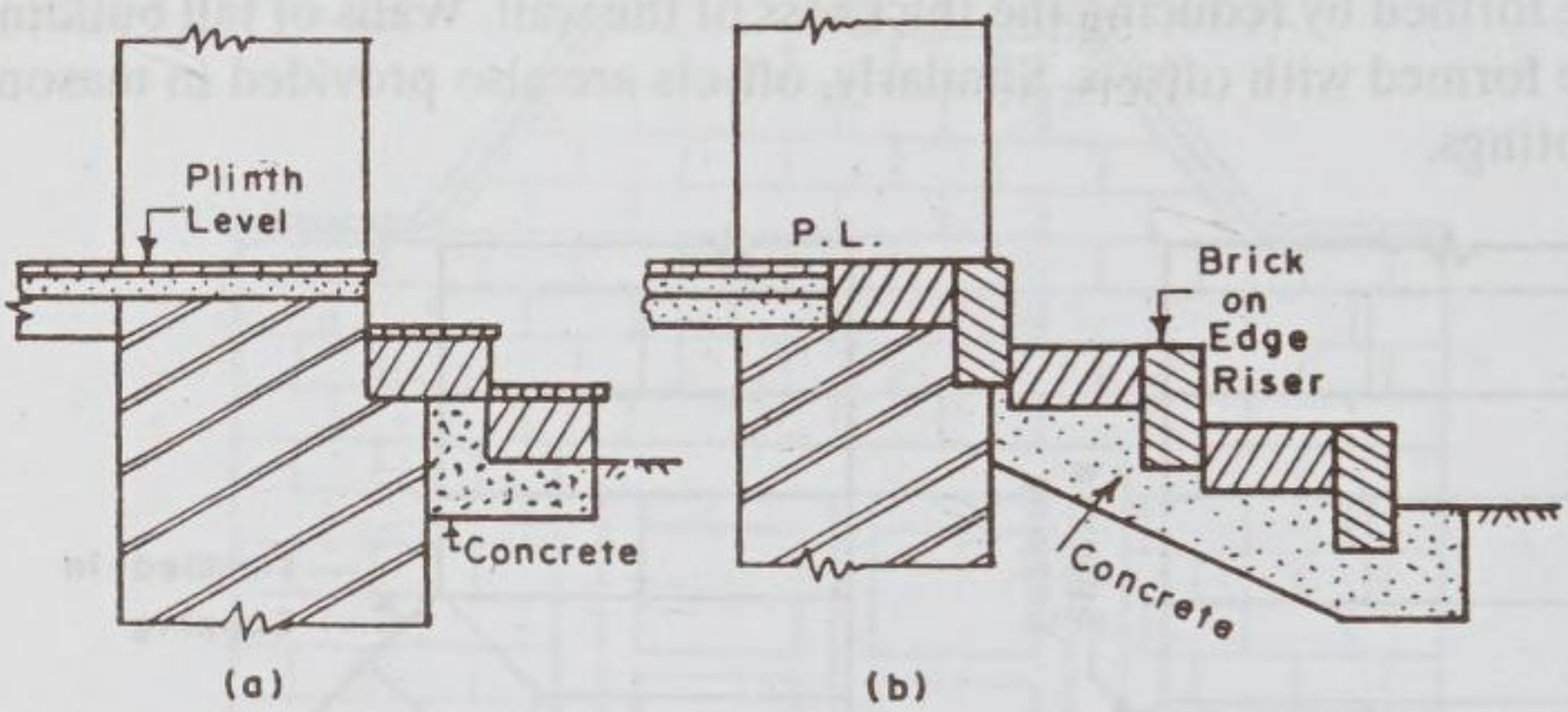


FIG. 5.13. THRESH HOLDS.

5.3. MATERIALS FOR STONE MASONRY

The following two materials are used for stone masonry:

1. Mortar
2. Stones.

(A) MORTAR

1. Definition and types

Mortar is a homogenious mixture, produced by uniform mixing of a *binder with inert material* (such as sand) and water to make a paste of required consistency and is used to bind a masonry unit. The following ingradienets are used for mortar making :

(*a*) Materials which cause adhesion when dried from wet plastic state such as clay, mud, etc., and

(*b*) Cementitious ingradients such as cement, lime or combination of these two, Portland pozzolana cement and lime-pozzolana mixture where sand is used as a fillter along with these binders to reduce the shrinkage characteristics of the mortar.

Choice of mortar and its grade for binding masonry units is governed by several considerations such as type of masonry, situations of use, load intensity, degree of exposure to weather, bond and durability reqirements, and other special considerations like fire resistance, insulation, rate of setting and hardening etc.

Cement used for preparing masonry mortars may be (*i*) ordinary portland cement, (*ii*) rapid hardening cement, (*iii*) blast furnace slag cement (*iv*) portland pozzolana cement, (*v*) masonry or trief cement.

If lime mortar is used, lime may be of hydraulic or semi-hydraulic category. However, where fat lime is used, it is essential

to add pozzolana such as burnt clay pozzolana or fly ash.

If mud mortar is used, the mud should be prepared from carefully selected soil of tenacious nature of sand content not less than 35 percent and plasticity index 8-10 for clayey soil and 6-10 for silty soils. In case suiable soil is not available, the blending of sand with clayey soil or *vice verse* may be done in suitable proportions so as to achieve the above physical characteristics of the soils. The sulphate content of such a selected soil shall not exceed 0.1 percent. Mud mortars are not preferred in stone masonry. It is sometimes used in brick masonry where low strength bricks are available and where the superimposed loads are not heavy.

2. Consistency of mortars

The quantity of water to be added to the mortar should be such that working consistency is obtained. Excess water should be avoided. In the case of cement lime mortars, the following formula may be used to get approximate quantity of water:

$$V_w = 0.65\,(W_c + W_l) \qquad ...(5.1)$$

where V_w = volume of water (in litres), per m^3 of sand

W_c = added amount of cement (in kg) per m^3 of sand.

W_l = added amount of lime (in kg) per m^3 of sand.

In general, the quantity of water depends upon the following factors:

(*i*) Nature and condition of fine aggregate,

(*ii*) Temperature and humidity at the time of working, and

(*iii*) Richness of the mix, i.e., whether richer or leaner than 1 : 3.

The working consistency of the mortar is usually judged by the mason during application. The water should be just enough to maintain the required fluidity of mortar during application. The consistency of the mortar to maintain required fluidity depends upon the joints of masonry. For example, thinner joints will require greater fluidity while joints subjected to heavy pressure intensity require stiffer mortar with less fluidity.

3. Lime mortar

Lime mortars are prepared from hydraulic and semi-hydraulic limes corresponding to class A and class B of IS : 712–1973. If fat limes corresponding to Class C is used, additiion of pozzolana is essential. Prepared lime mortars shall be kept damp and shall never be allowed to go dry. This may be ensured. Partly set or dried mortar shall never be retempered for use.

Strength of lime mortar depend upon mix proportions. Table 5.1 gives the compressive strength for various mix proportions.

TABLE 5.1
COMPERESSIVE STRENGTH OF MASONRY LIME MORTAR

S.No.	*Proportion of mix (by volume)*			*Compressive strength at 28 days*	
	Lime	*Pozzolana*	*Sand*	*Kg/cm²*	*N/mm²*
1.	1 (B)	—	3	5—7	0.5—0.7
2.	1 (A)	—	3	20—30	2—3
3.	1 (C)	—	1	30—50	3—5
4.	1 (C)	1	2	7—15	0.7—1.5
5.	1 (C)	2	—	30—50	3—5
6.	1 (C)	3	—	20—30	2—3

Note. (A), (B) and (C) denote the class of lime to be used, as specified in Indian Standard (IS:712–1973).

4. Cement mortar

The mortars with cement as an ingredient should be used as early as possible, preferably within half an hour from the time. Water is added to the cement during mixing operation or at the latest within one hour of its mixing. Cement mortars are generally more suitable for making high strength mortars. In addition to sand, pozzolana may also be added. Table 5.2 gives the compressive strength of cement mortars of various mix proportions :

TABLE 5.2
COMPRESSIVE STRENGTH OF CEMENT MORTARS

S.No.	*Mix proportion (by volume)*			*Compressive strength at 28 days*	
	Cement	*Pozzalana*	*Sand*	*Kgcm²*	*N/mm²*
1.	1	0	8	7—15	0.7—1.5
2.	1	0.4	8	7—15	0.7—1.5
3.	1	0	7	15—20	1.5—2.0
4.	1	0.4	7	15—20	1.5—2.0
5.	1	0	6	30—50	3.0—5.0
6.	1	0.4	6	30—50	3.0—5.0
7.	1	0	5	50 and above	5 and above
8.	1	0.4	5	—Do—	—Do—
9.	1	0	4	—Do—	—Do—
10.	1	0.4	4	—Do—	—Do—
11.	1	0	3	—Do—	—Do—
12.	1	0.4	3	—Do—	—Do—

5. Lime cement mortars (Gauged mortars)

The mortar in which cement is included as an ingredient in addition to lime is known as *gauged mortar or composite mortar.* The rate of stiffening of lime mortar is improved by gauging the lime with cement. Table 5.3 gives the compressive strength of gauged mortars of various mix proportions.

TABLE 5.3
COMPRESSIVE STRENGTH OF CEMENT MORTARS

S.No.	*Mix proportion by volume*				*Compressive strength at 28 days*	
	Cement	*Lime*	*Pozzolana*	*Sand*	*kg/cm²*	*N/mm²*
1.	1	3(B) or 4(C)	0	12	7—15	0.7—1.5
2.	1	2(B)	0	9	20—30	2—3
3.	1	1(C) or 1(B)	0	6	30—50	3—5
4.	1	3(C)	3	9	40—50	4—5
5.	1	0 to $\frac{1}{4}$(B) or (C)	0	4	50 and above	5 and above
6.	1	1(C)	2	4	—Do—	—Do—
7.	1	$\frac{1}{2}$ or $\frac{1}{4}$(C)	0	4.5	—Do—	—Do—

(B) STONES

The stones used for masonry should be hard, durable, tough and sound, and free from weathering, decay or defects like cavities, cracks, sand holes, injurious veins, patches of loose or soft materials etc. The stones should be obtained only from the approved quarry. The stone units should be obtained by quarrying large massive rock, and not by breaking small size boulders having rounded faces.

Rocks from which building stones are obtained, are divided into three groups : (1) Igneous (2) Sedimentary and (3) Metamorphic.

1. Igneous rock. These have been formed by agency of heat, the molten material subsequently become solidified. The chief building stone in this class is *granite*. Granite is hard and durable, and is used in steps, sills, facing work, walls etc. However, it is unsuitable for carving work. It is more suitable for heavy engineering works such as docks, break waters, light houses, masonry bridges and piers.

2. Sedimentary rocks.

These are those which have been formed chiefly through the agency of water. Most of these have been derived from breaking up of igneous rocks whose particles are conveyed and deposited by streams and accumulated to form thick strata that have been

subsequently hardened by pressure. The principal building stones in this group are *lime stones* and *sand stones.*

Lime stones. They consist of particles of carbonate of lime cemented together by a similar material. These are used in floors, steps and walls.

Sand stones. These are composed of consolidated sand and consist chiefly of grains of quartz (silica) united by cementing material. Sand stone is the most widely used building stone for steps, facing work, columns, walls etc.

3. Metamorphic rocks. These rocks form a group which embraces either igneous or sedimentary rocks which have been changed from their original form (or metamorphosed) by either pressure, or heat, or both. The common building stones that fall under this category are *slates and marbles. Slates* easily spilt along natural bedding planes. They are not very suitable for masonry work. They are used for roofing work, sills, damp-proof course etc. *Marbles* can take fine polish. Since they are costly, they are not used for masonry work.These are used for flooring, facing work, steps, ornamental work etc. Marbles can be easily sawn and carved.

Table 5.4 gives the recommendations for use of different types of stones for different purposes.

TABLE 5.4
RECOMMENDATIONS FOR TYPE OF STONES TO BE USED

Purpose	*Type of stone to be used*
1. Heavy engineering works such as docks, break waters, bridges, piers, etc., carry high intensity of pressure.	Fine grained granite and gneisses.
2. Masonry work in industrial area, exposed to smoke and chemical fumes.	Granite, compact sand stone, and quartzite
3. General building work.	Lime stone and sand stone.
4. Face work of buildings.	Marble, granite and closed grained sand stone
5. Carvings and ornamental work.	Marble, laterite and soft sand stone.
6. Pavings, door sills, steps.	Slate, sand stone, marble.
7. Fire resistant masonry.	Compact sand stone.

5.4. CLASSIFICATION OF STONE MASONRY

Depending upon the arrangement of stones in the construction, degree of refinement used in shaping the stone and finishing adopted, stone masonry can be classified as follows :

(*a*) *Rubble Masonry*

(*b*) *Ashlar Masonry.*

(A) RUBBLE MASONRY

In the rubble masonry, the blocks of stone that are used are either undressed or comparatively roughly dressed. The masonry has wide joints, since stones of irregular sizes are used. Rubble masonry may be out of the following types:

(*a*) *Random Rubble* — (*i*) Uncoursed. (*ii*) Built to courses.

(*b*) *Square Rubble* — (*i*) Uncoursed (*ii*) Built to courses. (*iii*) Regular coursed

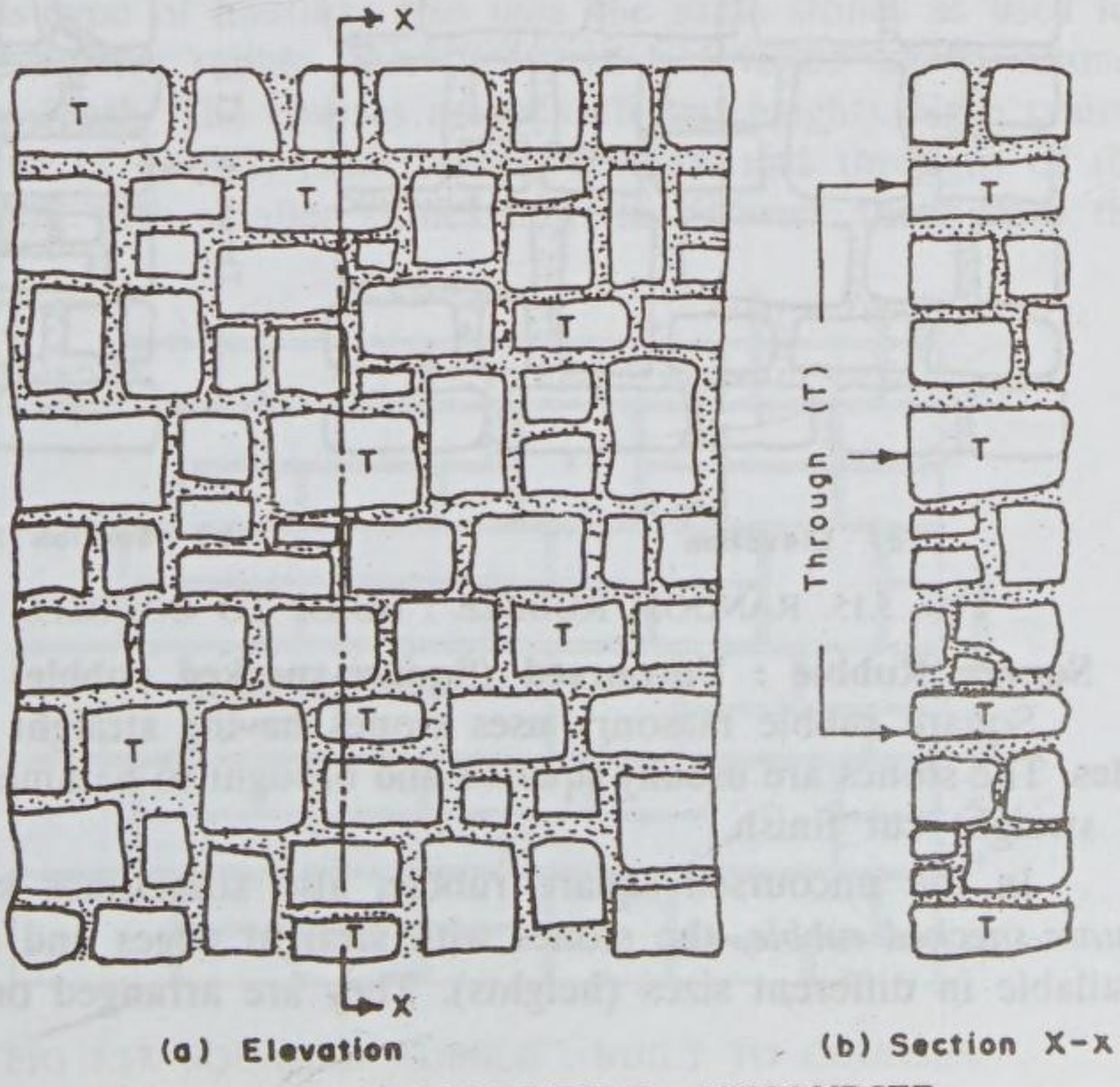
(*c*) *Miscellaneous types* — (*i*) Polygonal walling. (*ii*) Flint walling.

(*d*) *Dry rubble masonry.*

1. Random Rubble : Uncoursed

This is the roughest and cheapest form of stone walling. In this type of masonry, the stones used are of widely different sizes.

Since the stones are not of uniform size and shapes, greater care and ingenuity have to be exercised in arranging them in such a way that they adequately distribute the pressure over the maximum area and at the same time long continuous vertical joints are avoided.



(a) Elevation (b) Section X–x

FIG. 5.14. RANDOM RUBBLE : UNCOURSED.

Sound bond should be available both transversely as well as longitudinally. Transverse bond is obtained by the liberal use of *headers*. Larger stones are selected for quoins and jambs to give increased strength and better appearance. This type of masonry is also known as *uncoursed rubble masonry.*

2. Random Rubble : Built to Courses

The method of construction is the same as above except that the work is roughly levelled up to form courses varying from 30 to 45 cm thick. All the courses are not of the same height. For the construction of this type of masonry, quoins are built first and line (string) is stretched between the tops of quoins. The intervening walling is then brought up to this level by using different size of stones. Fig. 5.15 shows the procedure, in which the stone have been numbered in the order in which they are placed. This form of masonry is better than uncoursed random rubble masonry.

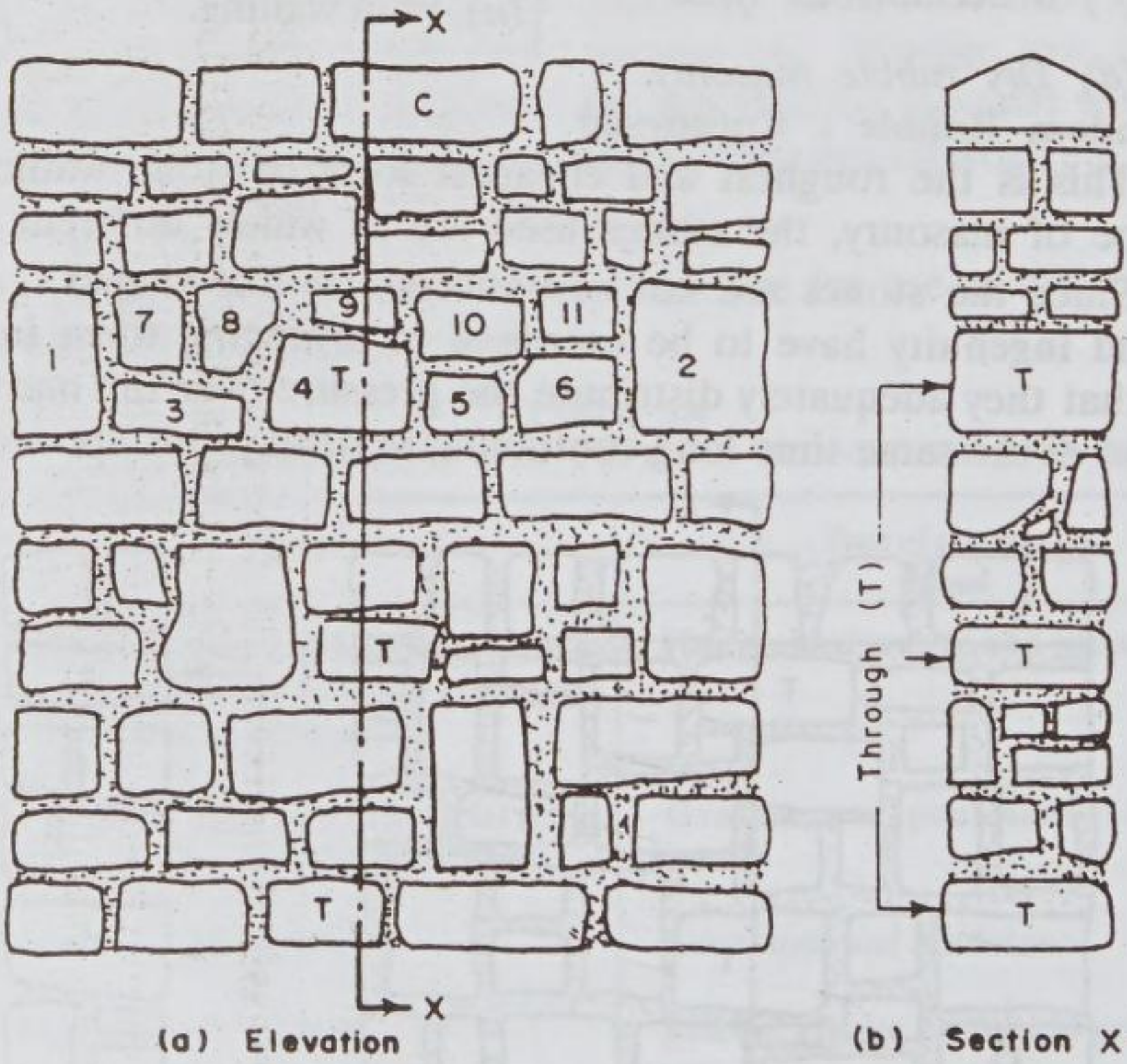


FIG. 5.15. RANDOM RUBBLE : BUILT TO COURSES.

3. Square Rubble : Uncoursed (Square-snecked rubble)

Square rubble masonry uses stones having straight bed and sides. The stones are usually squared and brought to hammer dressed or straight cut finish.

In the uncoursed square rubble, also sometimes known as *square-snecked rubble,* the stones with straight edges and sides are available in different sizes (heights). They are arranged on face in

several irregular pattern. Good appearance can be achieved by using *risers* (a large stone, generally a through stone), *leveller* (thinner stones) and *sneck or check* (small stone) in a pattern, having their depths in the ratio of 3 : 2 : 1 respectively. Snecks are the characteristics of this type of construction, and hence the name. This prevents the occurrence of long continuous joints.

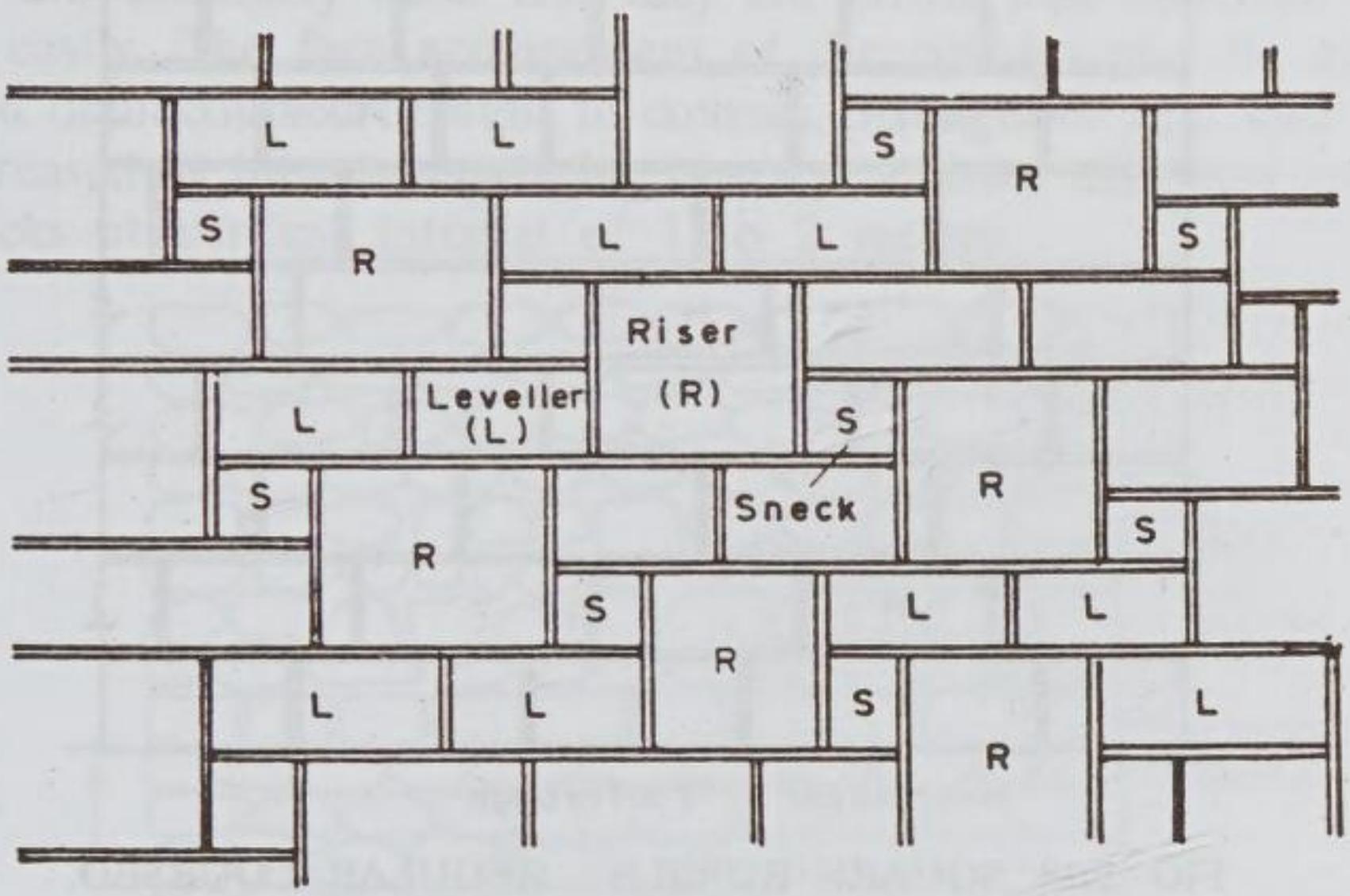


FIG. 5.16. UNCOURSED SQUARE RUBBLE.

4. Square Rubble : Built to courses

This type of masonry also uses the same stones as used for uncoursed square rubble. But the work is levelled up to courses of varying depth. The courses are of different heights. Each course may consist of quoins, jamb stones, bonders and throughs of the same height, with smaller stones built in between them upto the

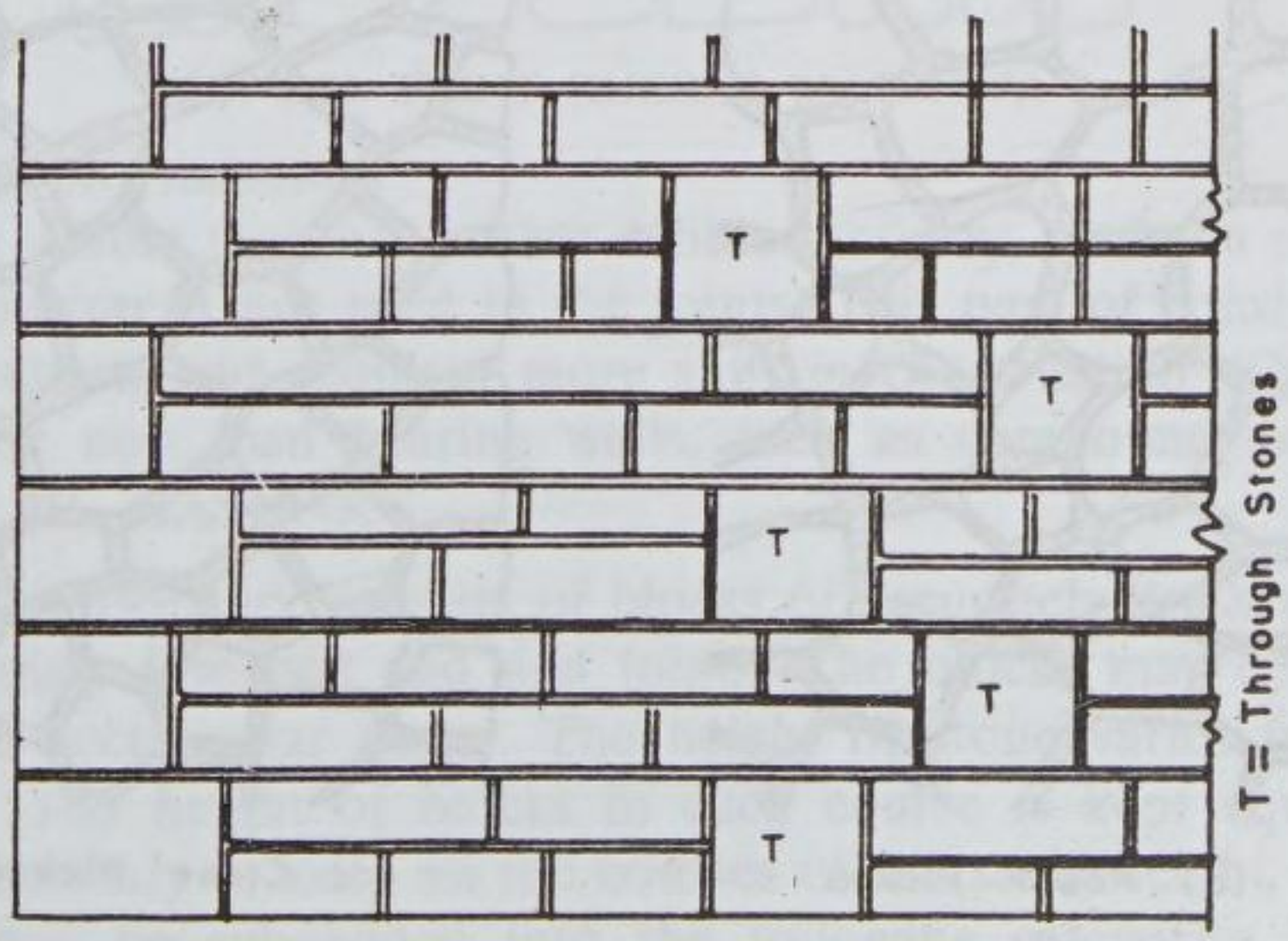


FIG 5.17. SQUARED RUBBLE : BUILT TO COURSES.

height of the larger stones, to complete the course.

5. Square rubble : Regular coursed

In this type of masonry, the wall consists of various courses of varying heights, but the height of stones in one particular course is the same.

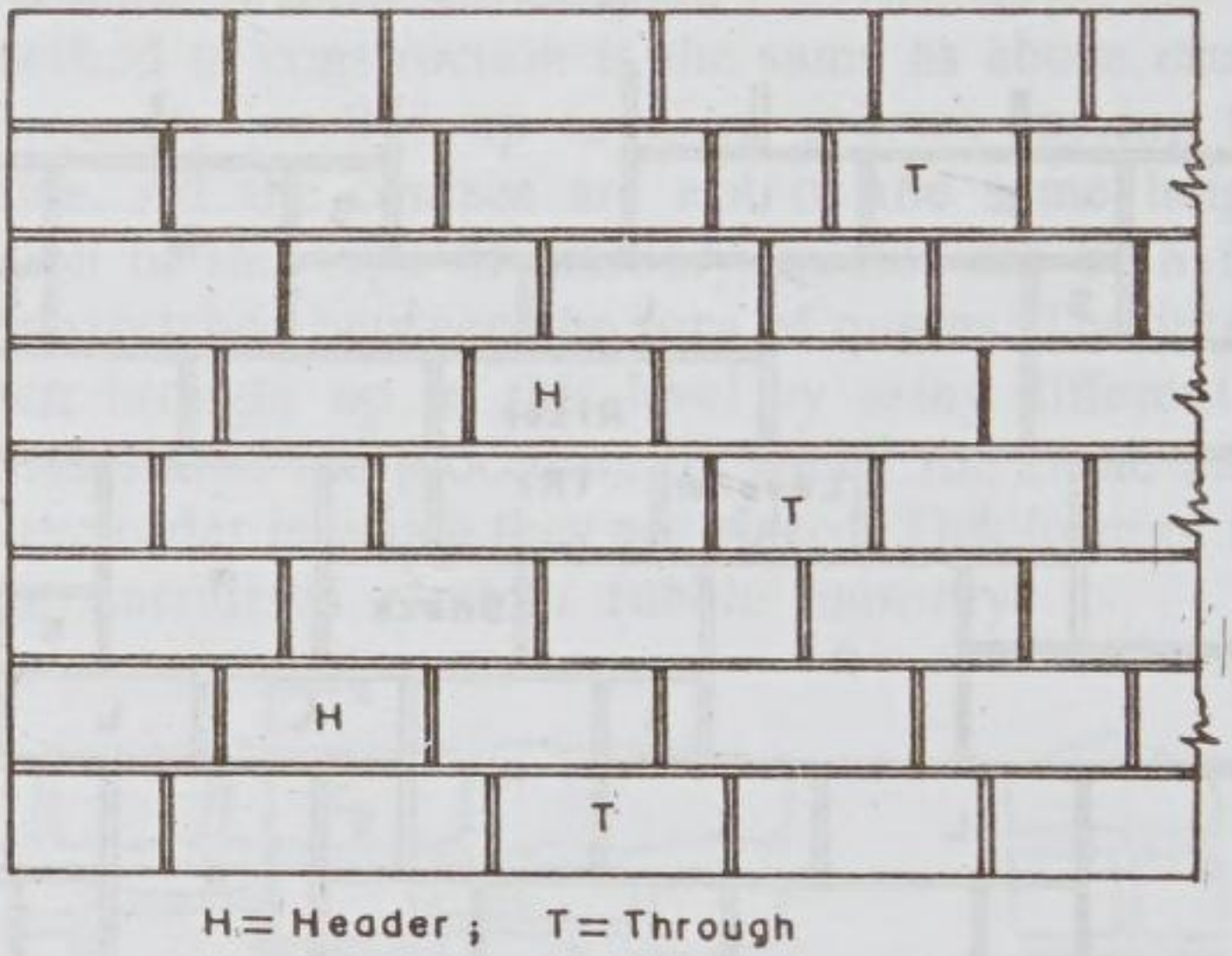


FIG. 5.18. SQUARE RUBBLE : REGULAR COURSED.

6. Polygonal Walling (Polygonal rubble masonry)

In this type the stones are hammer finished on face to an irregular polygonal shape. These stones are bedded in position to show face joints running irregularly in all directions. Two types of polygonal walling may be there : in the first type the stones are only roughly shaped, resulting in only rough fitting. Such a work is known as *rough picked*. In the second type, the faces of stones

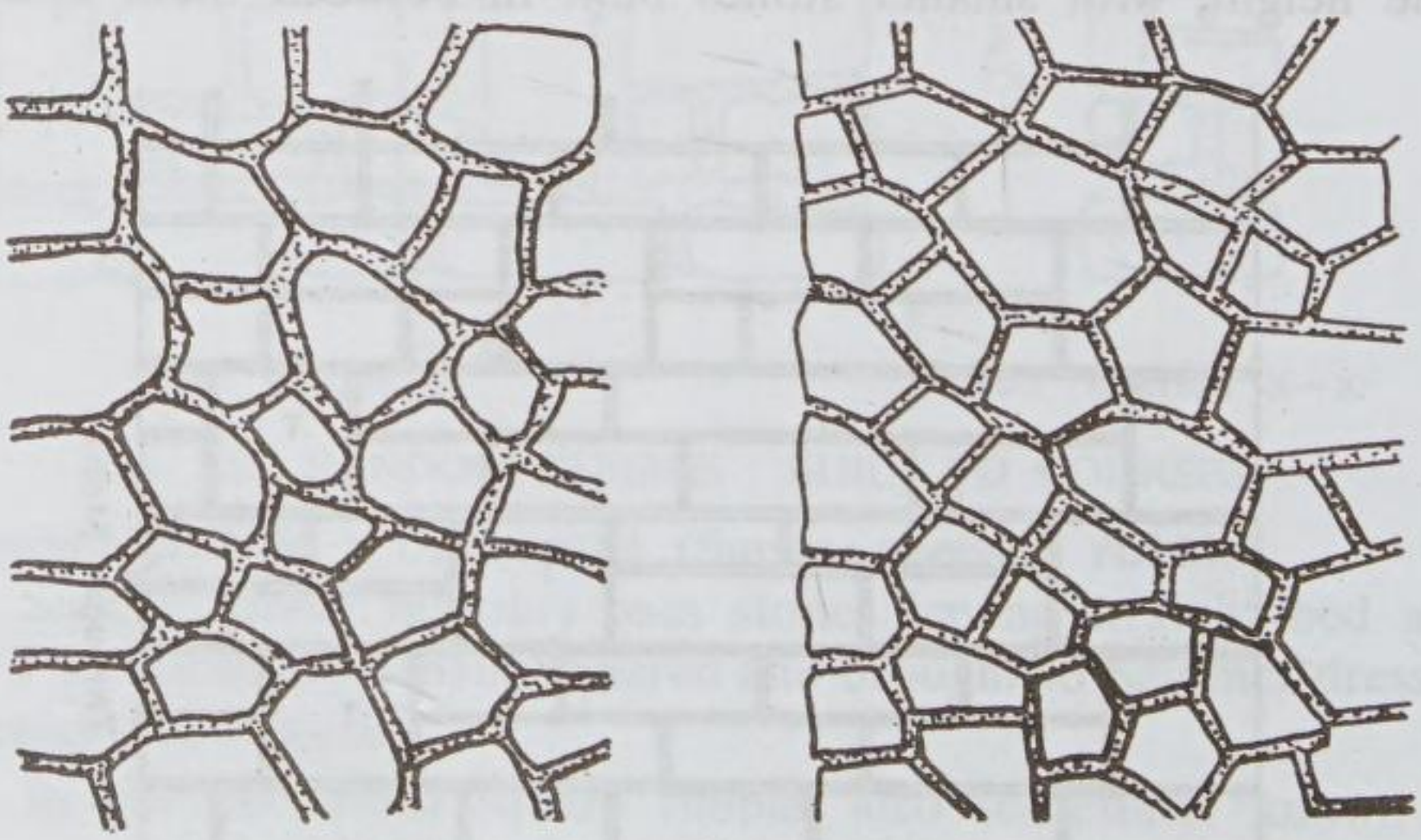


FIG. 5.19. POLYGONAL RUBBLE MASONRY.

are more carefully formed so that they fit more closely. Such a work is known as *close-picked work.*

7. Flint Walling (Flint rubble masonry)

The stones used in this masonry are flints or cobbles, which vary in width and thickness from 7.5 to 15 cm and in length from 15 to 30 cm. These are irregularly shaped nodules of silica. The stones are extremely hard. But they are brittle and therefore may break easily. The face arrangement of the cobbles may be either coursed or uncoursed or built to courses. Strength of flint wall may be increased by introducing lacing courses of either thin long stones or bricks at vertical interval of 1 to 2 metres.

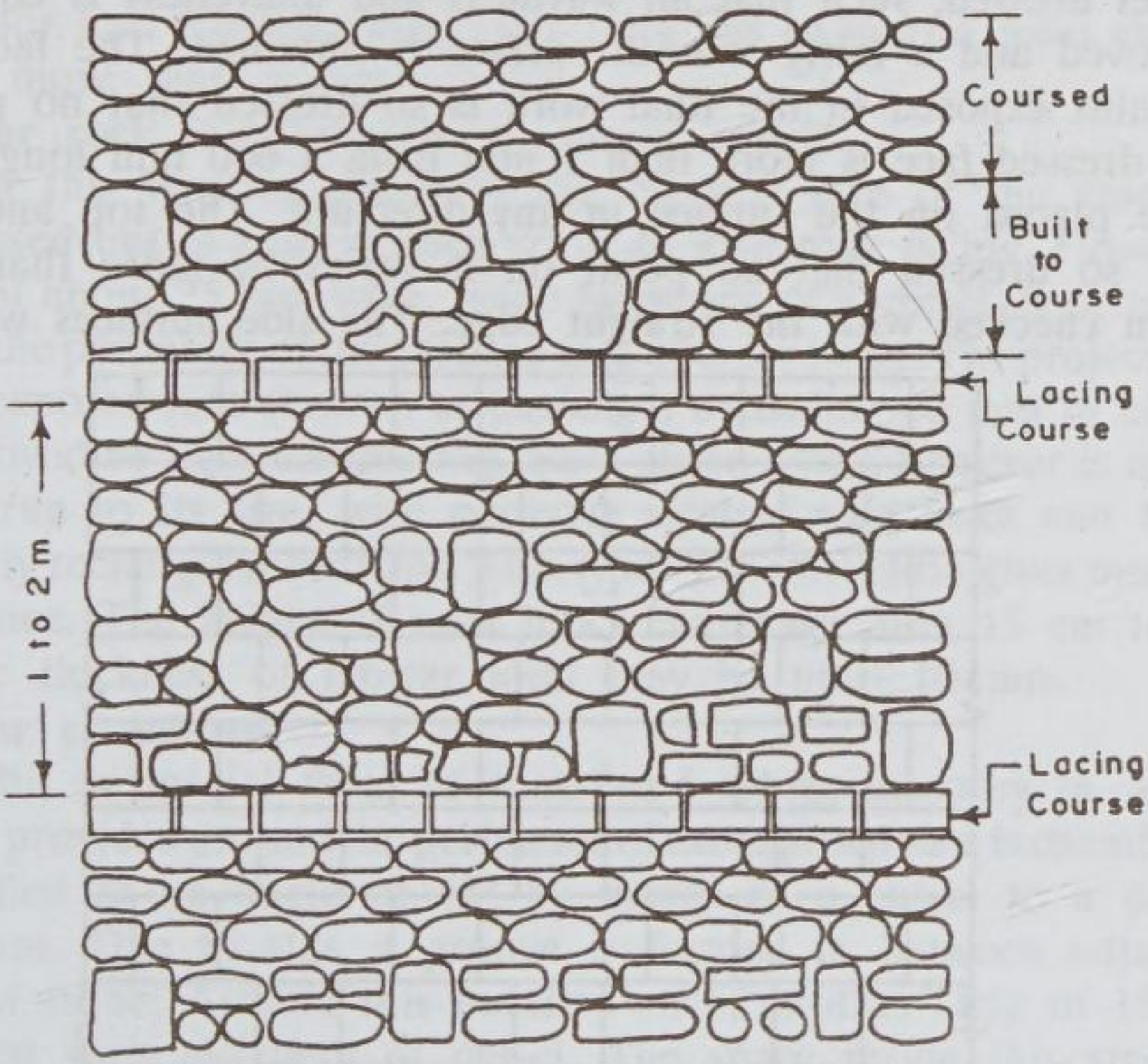


FIG. 5.20. FLINT RUBBLE MASONRY.

8. Dry rubble masonry.

Dry rubble masonry is that rubble masonry, made to courses, in which mortar is not used in the joints. This type of construction is the cheapest, and requires more skill in construction. This may be used for non load bearing walls, such as compound wall etc.

(B) ASHLAR MASONRY

Ashlar masonry consists of blocks of accurately dressed stone with extremely fine bed and end joints. The blocks may be either square and rectangular shape. The height of stone varies from 25 to 30 cm. The height of blocks in each course is kept equal but it is not necessary to keep all the courses of the same height. Ashlar masonry may be subdivided into the following categories :

(1) Ashlar fine tooled
(2) Ashlar rough tooled
(3) Ashlar rock, rustic or quarry faced
(4) Ashlar chamfered
(5) Ashlar block in course
(6) Ashlar facing

1. Ashlar fine tooled

This is the finest type of stone masonry work. Each stone is cut to regular and required size and shape so as to have all sides rectangular, so that the stone gives perfectly horizontal and vertical joints with adjoining stone. The beds, Joints and faces are chisel dressed, such that all waviness and uneveness is completely removed and a fairly smooth surface is obtained. The face which remains exposed in the final work is so dressed that no point on the dressed face is more than 1 mm from a 600 mm long straight edge placed on the surface in any direction. The top and bed is also so dressed that no point on it varies by more than 3 mm when checked with the straight edge. The side surfaces which are

FIG. 5.21. FINE TOOLED ASHLAR MASONRY.

to form the vertical joints are also so dressed that no point on the surface is more than 6 mm from the straight edge. The surfaces forming internal joints which are not visible are also so dressed that no point on the surface is more than 10 mm from the straight edge. All angles and edges that remain exposed in the final position are kept as true square and free from chipplings. The thickness of courses is generally not less than 15 cm. The width of stone is not kept less than its height. Headers and stretchers are laid alternately in each course or course of headers and course of stretchers may be laid alternately or they may be laid as otherwise directed. The thickness of mortar joint is kept uniform throughout and it

should not be more than 5 mm. The exposed joints are finely pointed.

2. Ashlar rough tooled (Bastard ashlar)

In this type of masonry, the beds and sides of each stone block are finely chisel dressed just in the same manner as for ashlar fine, but the exposed face is dressed by rough tooling. A strip, about 25 mm wide and made by means of a chisel is provided around the perimeter of the rough dressed face of each stone. The rough tooled face when tested with a straight edge 600 mm in length, should not show any point on the surface to vary by more than 3 mm in any direction. This type of masonry is also known as *bastard ashlar*. The size, angle, edges etc. are maintained in order, similar to that for fine dressed ashlar. The thickness of mortar joint should not be more than 6 mm.

3. Ashlar rock faced (rustic or quarry faced)

In this type of masonry, the exposed face of the stone is not dressed but is kept as such so as to give rock facing. However, a strip of about 25 mm wide, made by means of a chisel, is provided around the perimeter of the exposed face of every stone. The projections on the exposed face (known as *bushings*) exceeding 80 mm in height are removed by light hammering. Each stone block, however is maintained true to its size, with perfectly straight side faces and beds, and truely rectangular in shape. This type of construction gives massive appearance. The height of each block may vary from 15 cm to 30 cm. The thickness of mortar joint may be upto 10 mm.

4. Ashlar chamfered

This is special form of rock-faced ashlar masonry in which the strip provided around the perimeter of the exposed face is chamfered or bevelled at an angle of 45° by means of a chisel to a depth of 25 mm. Due to this, a groove is formed in between adjacent blocks of stone. Around this bevelled strip, another strip of 15 cm is dressed with the help of chisel. The space inside this strip is

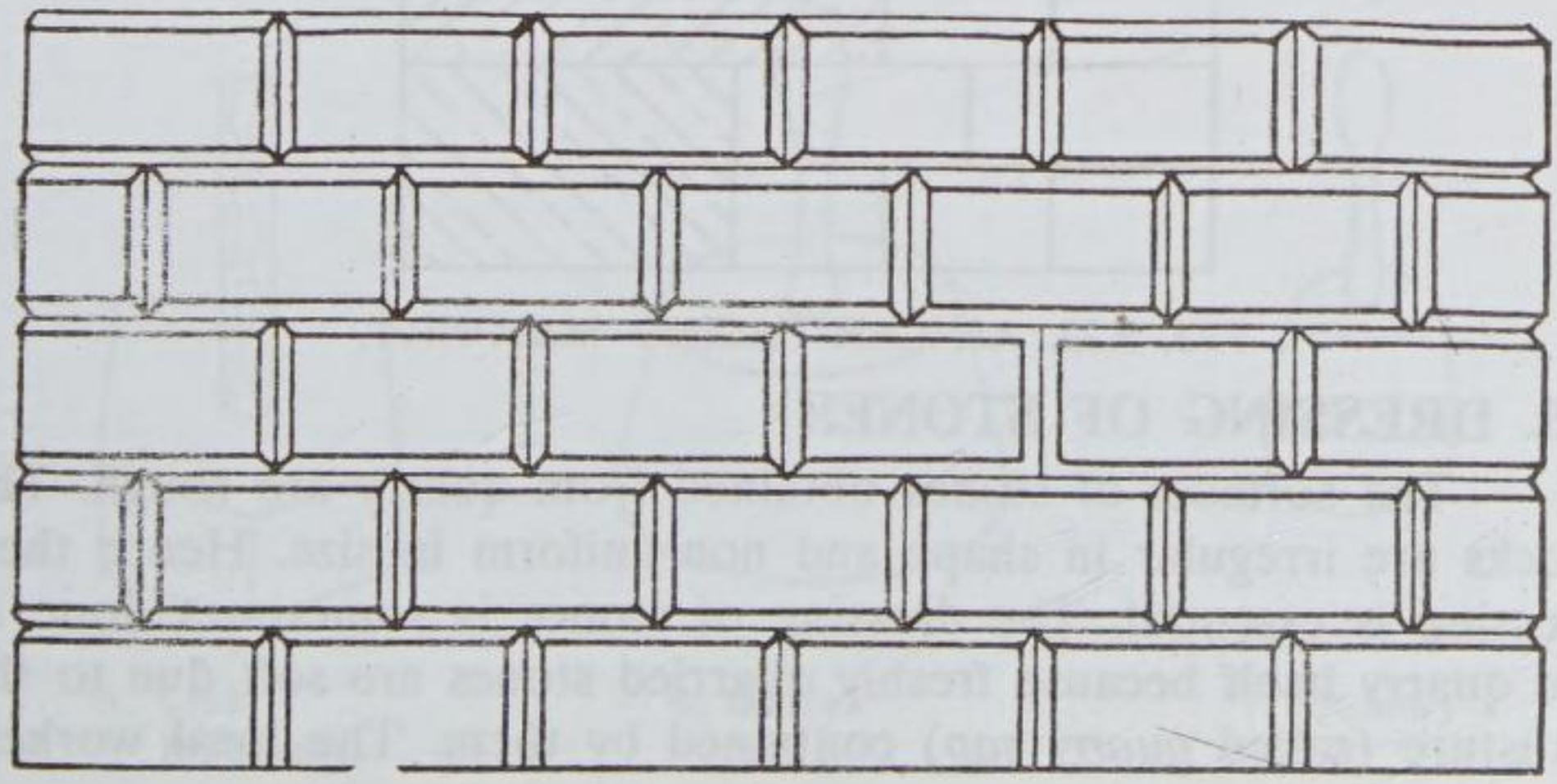


FIG. 5.22. ASHLAR CHAMFERED.

kept rock faced except that large bushings in excess of 80 mm projections are removed by a hammer.

5. Ashlar block in course

This type of masonry is intermediate between rubble masonry and ashlar masonry. The faces of each stone are hammer dressed, and the height of blocks is kept the same in any course, though it is not necessary to keep uniform height for all the courses. The vertical joints are not as straight and as fine as in ashlar masonry. The depth of courses may vary from 15 to 30 cm. This type of masonry is adopted in heavy works such as retaining walls, bridges etc.

6. Ashlar facing

Ashlar facing masonry is provided along with brick or concrete block masonry, to give better appearance. The sides and beds of each block are properly dressed so as to make them true to shape. The exposed faces of the stone are rough tooled and chamfered. The backing of the wall may be made in brick masonry.

FIG. 5.23. ASHLAR FACING MASONRY.

5.5. DRESSING OF STONES

The surfaces of stones obtained from quarry are rough. The blocks are irregular in shape and non-uniform in size. Hence their dressing is essential. The dressing of stones is sometimes done at the quarry itself because freshly quarried stones are soft due to the moisture (called *quarry sap*) contained by them. The local workers

are more experienced in the art of dressing of that particular type of stone. Also, if the stones are dressed at the quarry site itself, the trasportation costs are reduced because of reduction in the weight due to dressing.

Tools and implements for stone dressing

The following tools are used for dressed stones (Figs. 5.24 and 5.25).

1. *Spall hammer* : Used for rough dressing of stones.
2. *Scrabbling hammer* : Used for removing irregular bushings
3. *Mash hammer* : Used for rough dressing.
4. *Wallers hammer* : Used for removing spalls.
5. *Club hammers* : Used to strike narrow-headed chisels.
6. *Mallet* : Used to strike mallet headed chisels.
7. *Dummy* : Used for striking chisels for carving work.
8. *Gad* : Used to split stones
9. *Drag* : Used to give drag finish.
10. *Hand saw* : Used to cut soft stones.
11. *Cross-cut saw* : Used to cut hard stones.
12. *Frame saw* : Used to cut large blocks of stones.
13. *Pitching tool* : Used to make stones of required size.

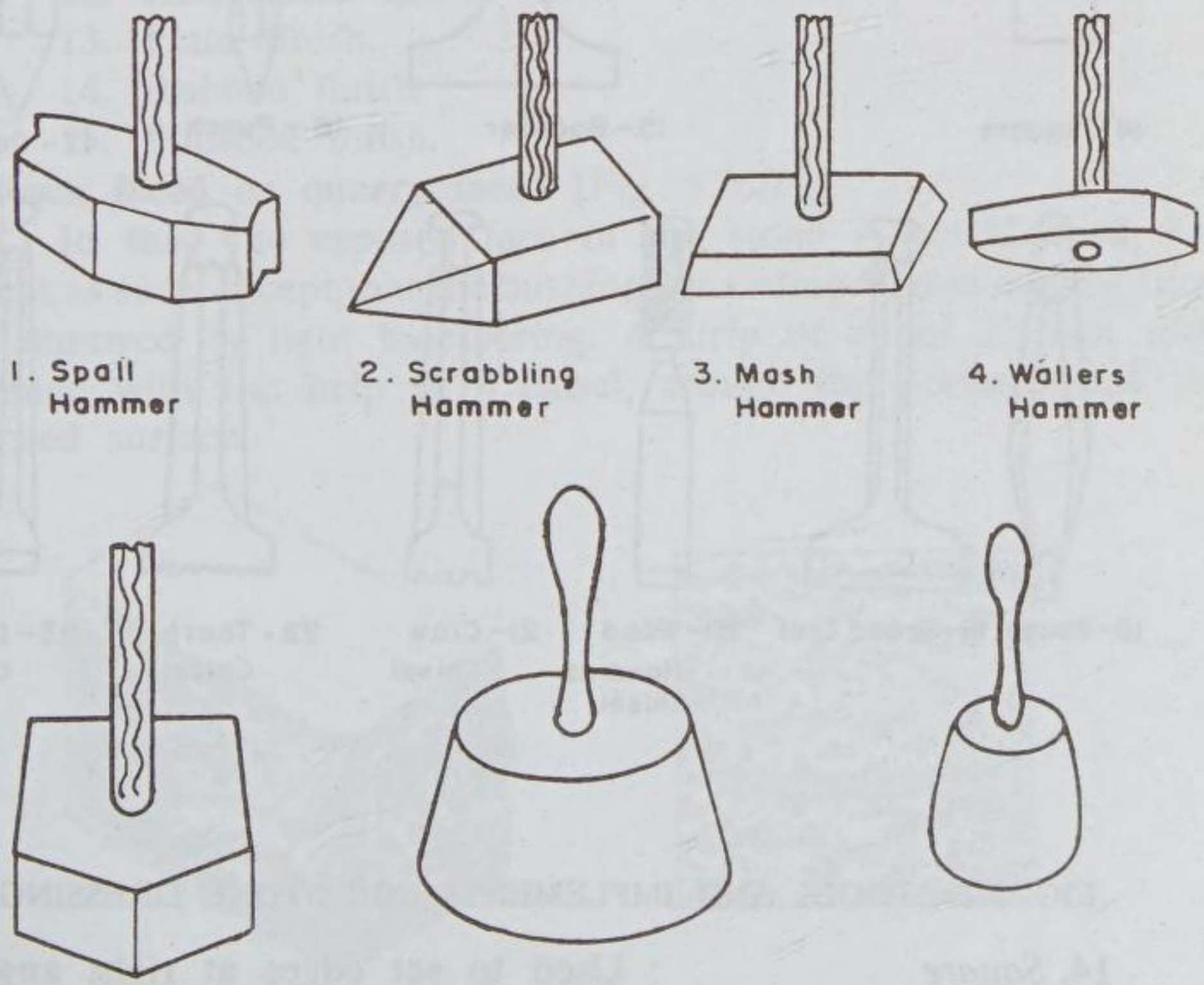
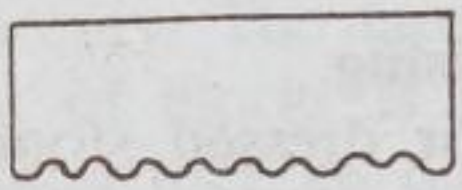
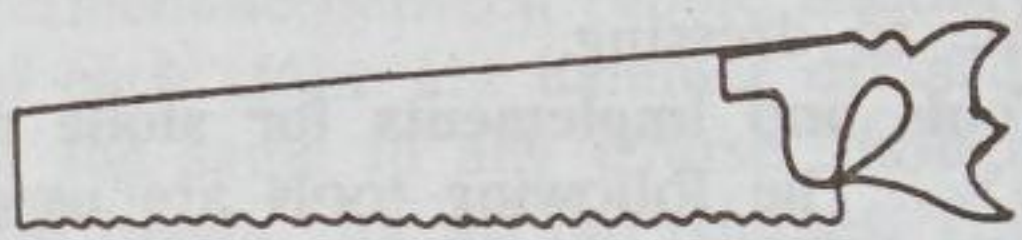


FIG. 5.24. TOOLS AND IMPLEMENTS FOR STONE DRESSING.

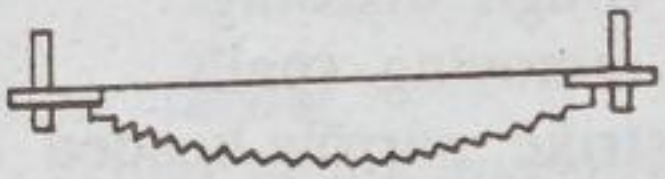
8 – Gad



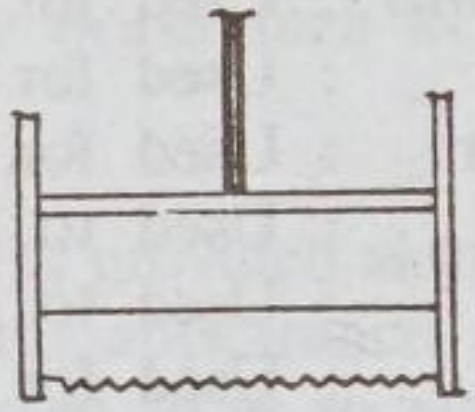
9 - Drag



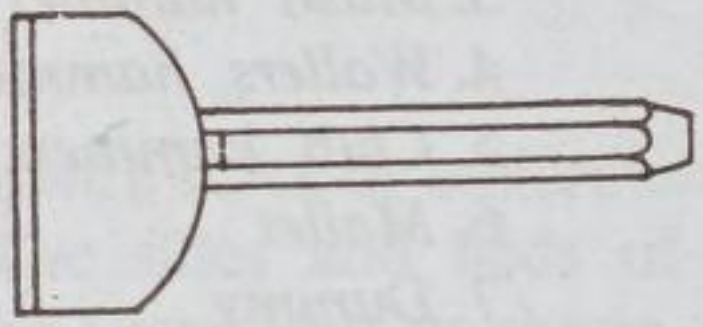
10 - Hand Saw



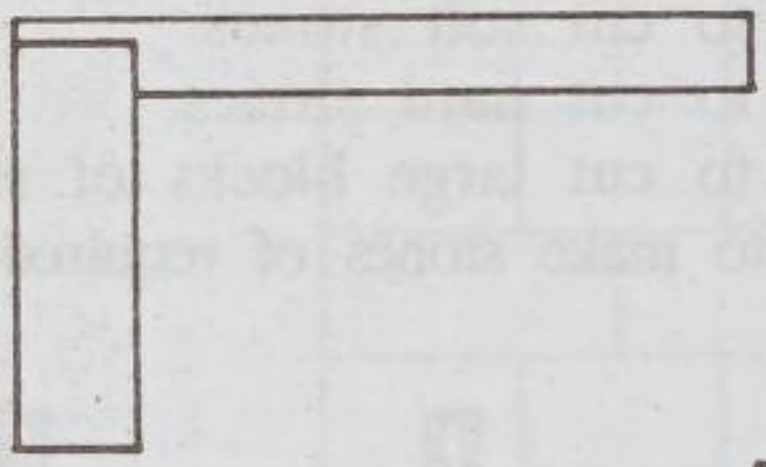
11 - Cross-Cut Saw



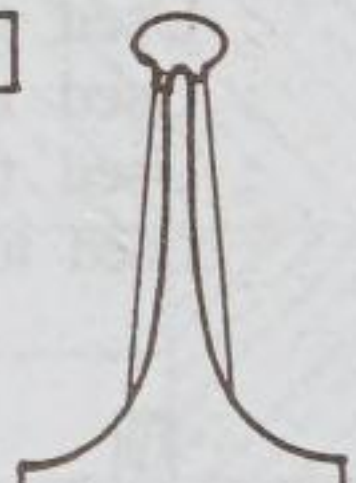
12 - Frame Saw



13 - Pitching Tool



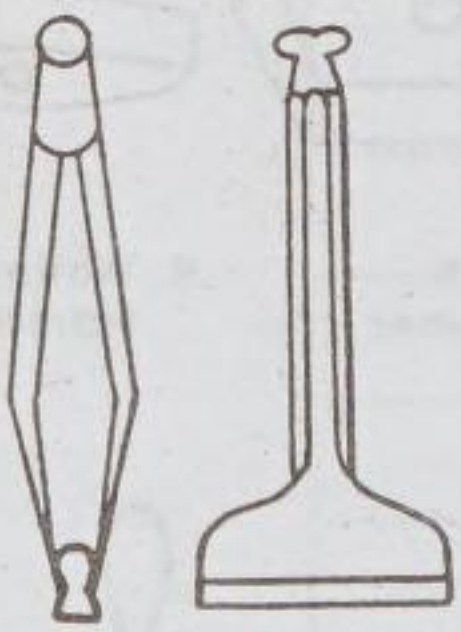
14 - Square



15 - Boaster

16 – Punch

17 - Point

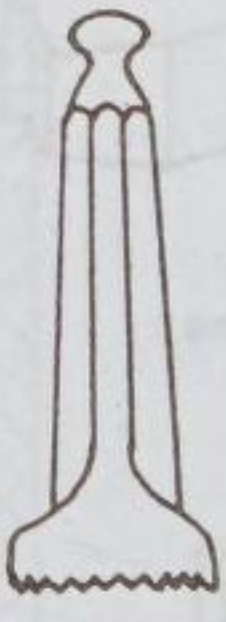


18-Gouge

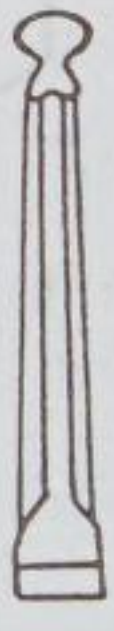
19-Broad Tool

20-Wood Handled Chisel

21- Claw Chisel



22- Tooth Chisel



23- Drafting Chisel

FIG. 5.25. TOOLS AND IMPLEMENTS FOR STONE DRESSING.

14. *Square* : Used to set edges at right angles.
15. *Boaster* : Used to cut soft stones.
16. *Punch* : Used for rough dressing.
17. *Point* : Used for rough dressing of hard stones.

18. *Gouge* : Used to dress stones for cornices, string courses etc.
19. *Broad tool* (nicker) : Used to form chisel lines on stone surface.
20. *Wood handled chisel* : Used to dress soft stones.
21. *Claw chisel* : Used to dress hard stones.
22. *Tooth chisel* : Used to dress hard stones.
23. *Drafting chisel* : Used for fine dressing.

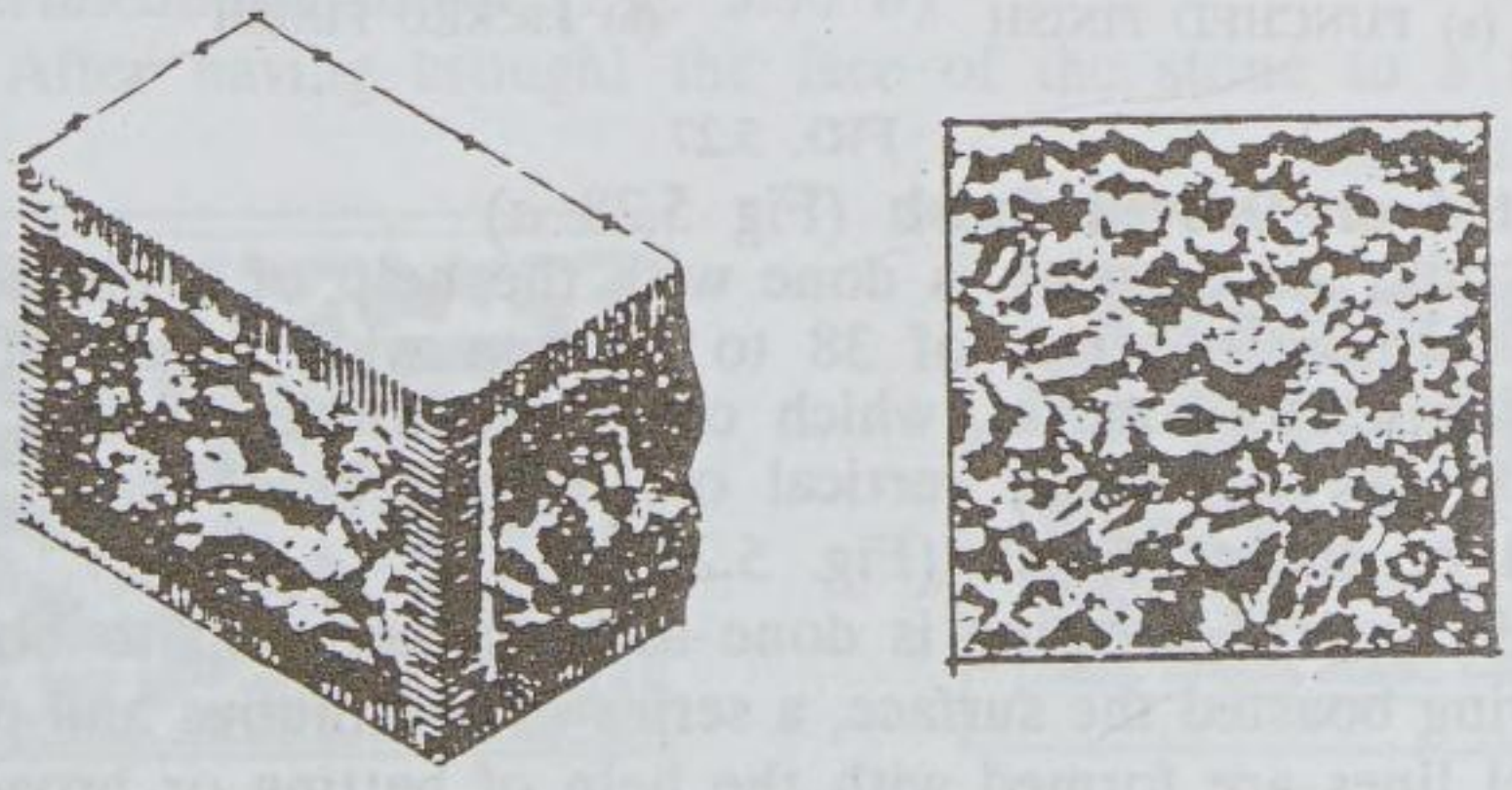
Types of surface finishes .

Dressed stones may have following types of surface finishes

1. Rock-faced or quarry faced finish.
2. Scabbling finish.
3. Hammer dressed finish.
4. Axed finish.
5. Punched finish.
6. Picked finish.
7. Boasted finish.
8. Tooled finish.
9. Furrowed finish.
10. Dragged or combed finish.
11. Vermiculated finish.
12. Reticulated finish.
13. Plain finish.
14. Rubbed finish.
15. Polished finish.

1. Rock faced or quarry faced [Fig. 5.26(*a*)]

In this, the exposed face of the stone is not dressed, but is kept as such, except that the bushings exceeding 80 mm in projection are removed by light hammering. A strip of about 25 mm wide is made with the help of a chisel, around the perimeter of the exposed surface.



(a) ROCK FACED (b) HAMMER FINISH

FIG. 5.26

2. Scabbling finish

This is type of rough dressing in which the irregular projections are removed by a scabbling hammer.

3. Hammer-dressed finish (Fig. 5.26 *b*)

The stone blocks are made roughly square or rectangular by means of Waller's hammer. The exposed face is roughly shaped by means of a mash hammer. The beds and joints are dressed back some 75 to 100 mm from the face. This is done by using the square to mark the boundaries and using pitching tools along the boundaries.

4. Axed finish

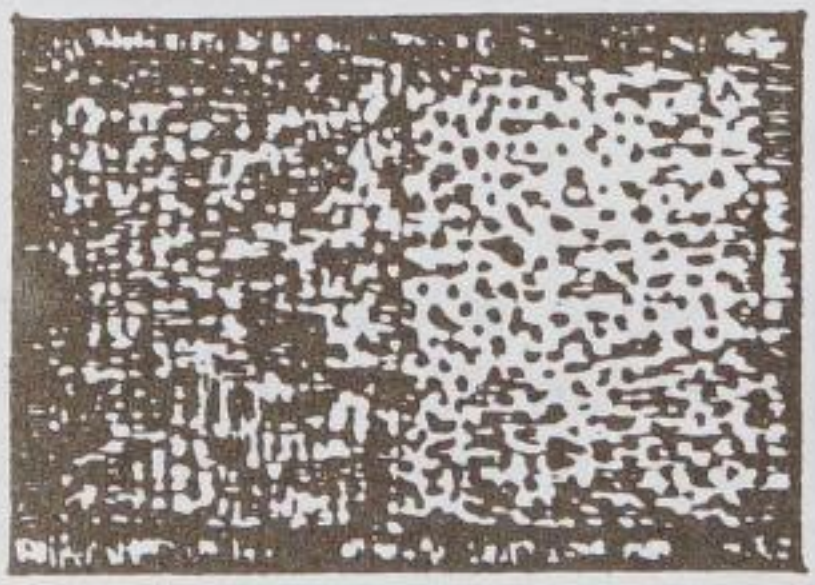
This type of finishing is used in hard stones like granites, where the dressing is done with the help of an axe.

5. Punched, Broached or Stugged finish (Fig. 5.27 *a*).

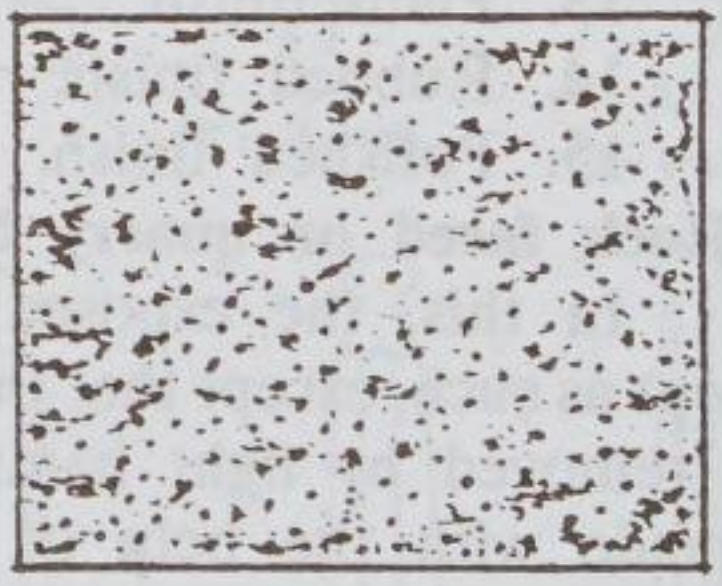
This is another form of rough dressing, usually used for lower portions of the buildings. The exposed face of the stone is dressed with the help of a punch, thus making depressions or punch holes on at some regular distance (say 25 mm) apart. A 25 mm wide strip is made around the perimeter of the stone with the help of chisel.

6. Picked finish (Fig.5.27 *b*)

This is similar to the above except that a point is used in the place of punch, thus forming small pits on the exposed face.



(a) PUNCHED FINISH



(b) PICKED FINISH

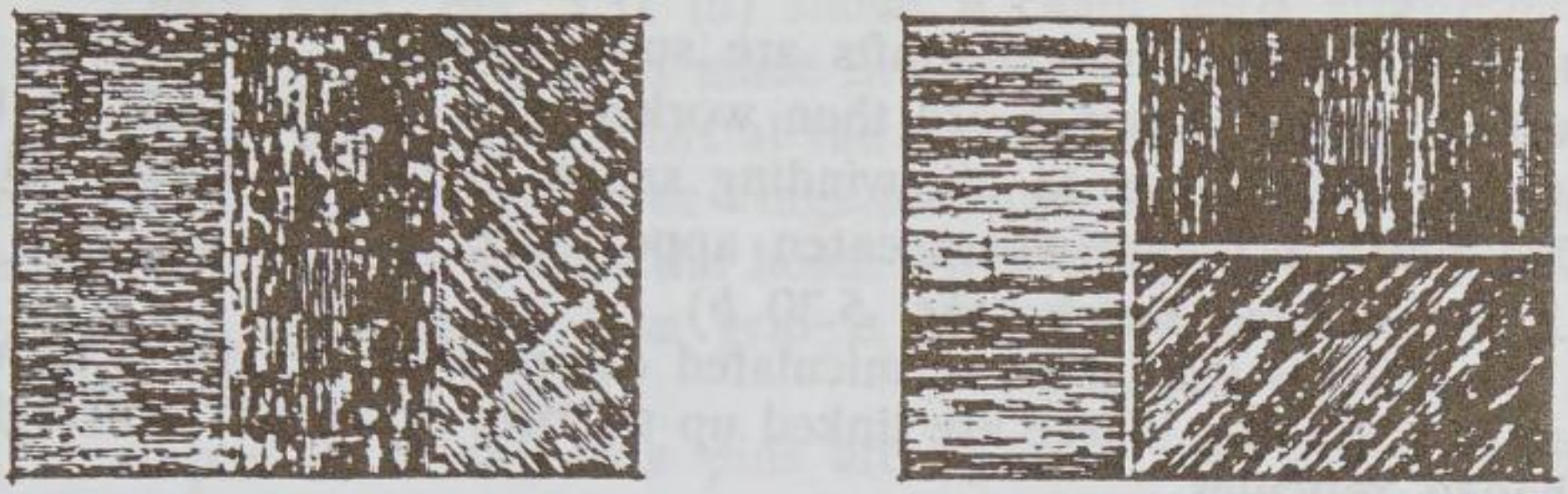
FIG. 5.27

7. Boasted or Droved finish (Fig 5.28 *a*)

In this, the dressing is done with the help of a boaster and hammer, forming a series of 38 to 50 mm wide *bands* of more or less parallel tool marks, which cover the whole surface. These marks may be horizontal, vertical or inclined at 45°.

8. Tooled or Batted finish (Fig. 5.28 *b*)

This type of dressing is done as a further step to boasting. After having boasted the surface, a series of continuous and parallel fine chisel lines are formed with the help of batting or broad tool. *This is common dressing for ashlar work.* The lines are deeper and continuous.



(a) BOASTED FINISH (b) TOOLED FINISH

FIG. 5.28

9. Furrowed finish (Fig. 5.29)

This type of finish is applied to the fillets or flat bands of cornices, string courses, doors and windows, architraves etc. After boasting the surface and then rubbing it, 6 to 10 mm wide flutes are formed by a gauge. A margin of about 20 mm width is sunk on all the edges of the stones and the central portion is made to project about 15 mm.

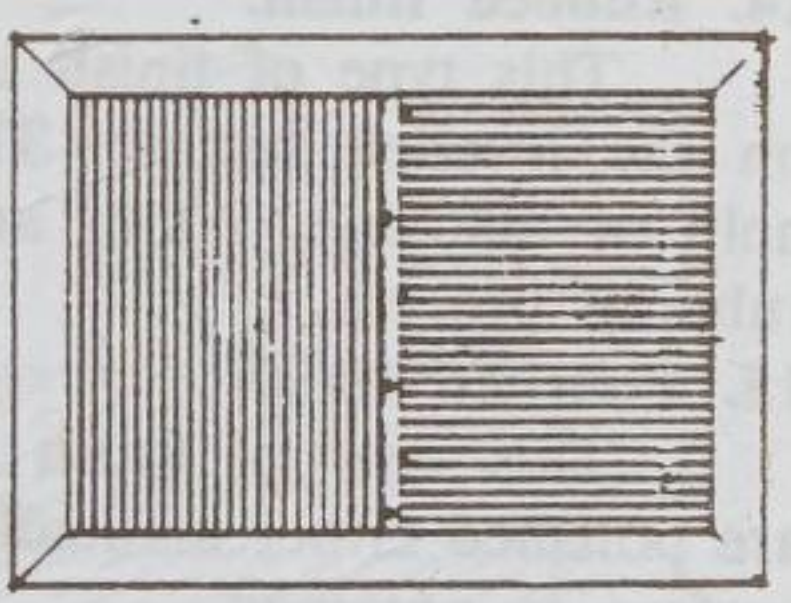


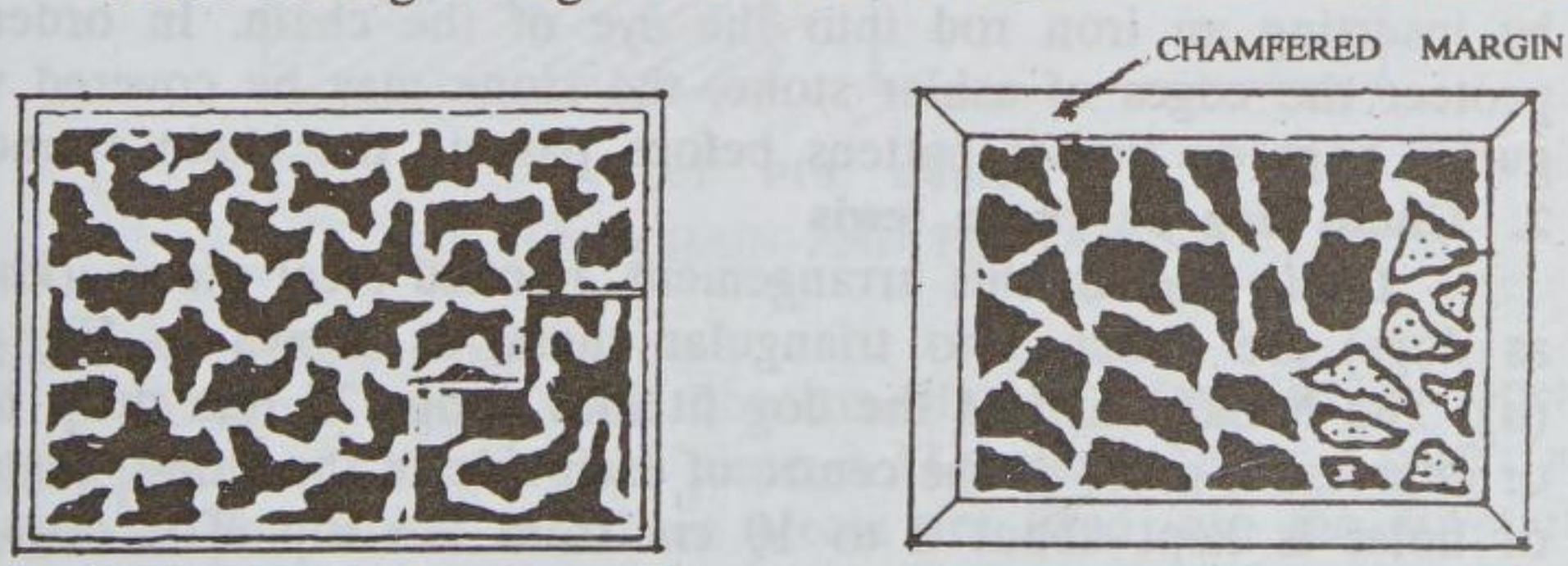
FIG. 5.29. FURROWED FINISH.

10. Dragged or Combed finish

This finish is used only in soft stones. The surface of the stone is first brought to the required level by means of a *dummy* (the head of which is made of zinc) and soft stone chisel. Drags, made of steel plates and of different grades (*i.e.* coarse, medium, fine) are then dragged backward and forward in different directions until the tool marks are eliminated. Fine drag is used at the end, which eliminates all the scratches on the stone. A combed finish is obtained.

11. Vermiculated finish (Fig. 5.30 *a*)

After having brought the face of the stone to a level and



(a) VERMICULATED (b) RETICULATED

FIG. 5.30.

smooth finish, marginal drafts are sunk about 10 mm below the surface. These sinkings are then worked to a depth equal to that of the drafts so as to cut winding snake like (verminous) ridges. The finish presents worm eaten appearance.

12. Reticulated finish (Fig. 5.30 *b*)

This is similar to vermiculated except that the ridges or vein are less winding. These are linked up to form polygonal or irregluar shaped reticules.

13. Plain finish.

In this type of finish, the surface is smade approximately smooth with a saw or a chisel.

14. Rubbed finish.

This type of finish is obtained by rubbing a piece of stone on the levelled surface. The rubbing can also be done with the help of machine. Water and sand may be used to accelerate the rubbing process.

15. Polished finish

This type of finish is used in marbles, granites etc. These are polished either manually or with the help of machines. A glossy surface is obtained.

5.6 APPLIANCES FOR LIFTING STONES

Small stone units are usually lifted by hand and then placed in position during masonry construction. However, big stone units and stone blocks are lifted by the following lifting appliances:

1. Chain or rope
2. Chain dog and chain lewis
3. Pin lewis
4. Three legged lewis
5. Nippers.

1. Chain or rope.

This is the simple method, usually adopted for lifting stone slabs. Chain is wrapped round the stone and then tied firmly to it. The chain is then connected to pulley blocks, or even lifted manually by inserting an iron rod into the eye of the chain. In order to protect the edges of ashlar stone, the stone may be covered with gunny bags or timber battens before passing the chain round it.

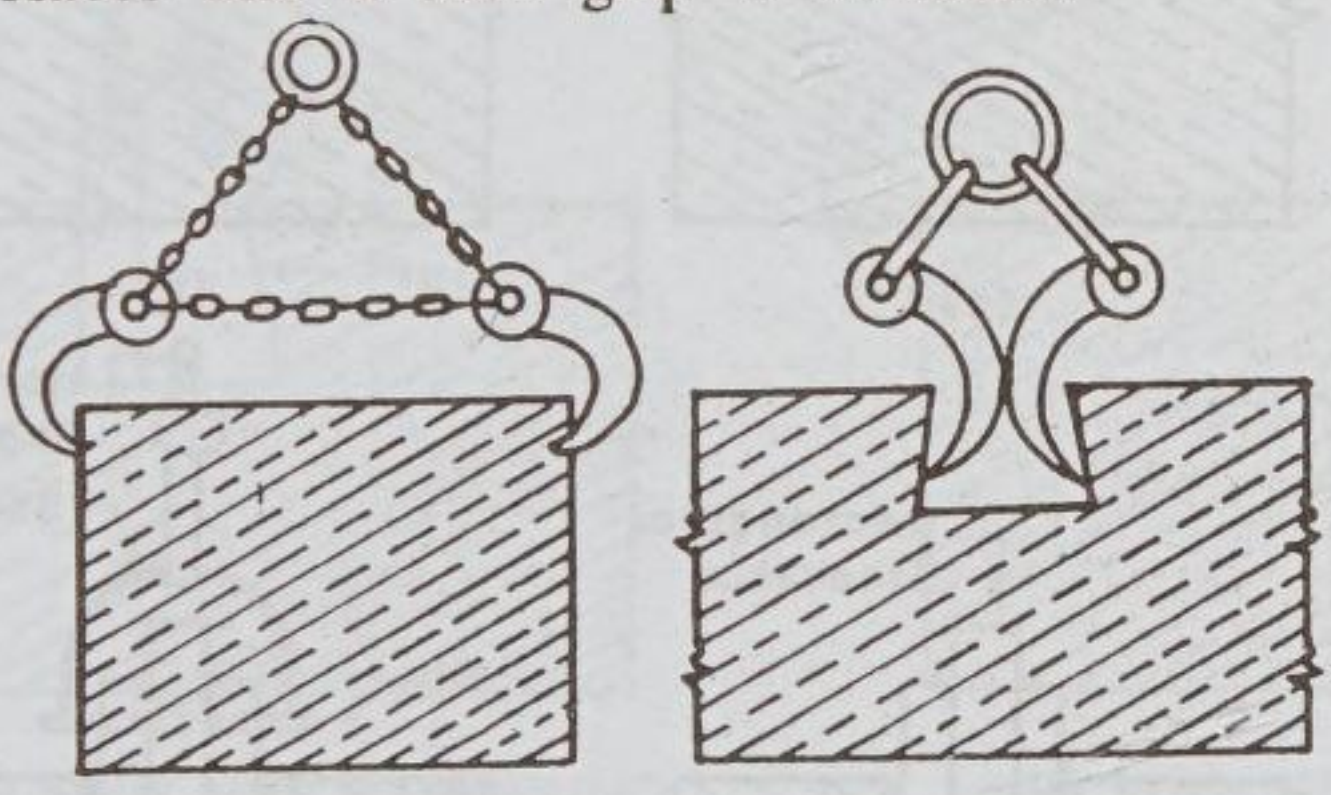
2. Chain dog or chain lewis

Chain dog. In this arrangement, hooked steel pieces (known as dogs) are attached to triangular chain, as shown in Fig. 5.31 (*a*). The hooked end of the dog fit into about 20 mm deep holes or depression made at the centre of each side of the stone. Distance of holes is kept about 8 to 10 cm from the top. The dogs are connected to the hoisting chain. When the crane chain is wound up, the dogs or hooks bite into the stone; due to this a firm grip is obtained.

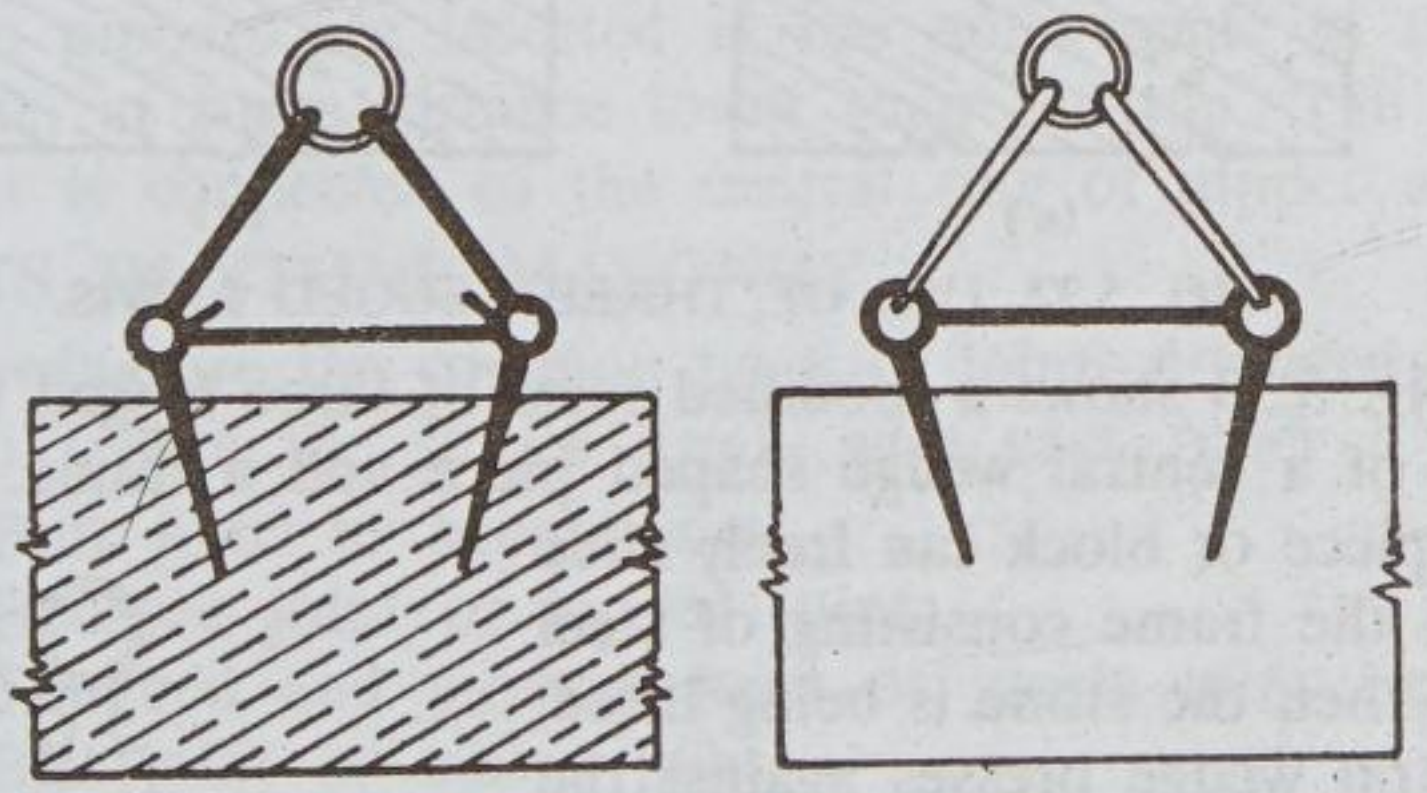
Chain lewis. Fig. 5.31 (*b*) shows a chain lewis arrangement in which a dovetailed hole is made in the centre of the top surface of the stone. The hole is short at the top and wider at the bottom. Chain lewis consists of three rings and two curved pins or legs. The curved pins are put in the hole. While lifting, the pins tighten against the hole and a firm grip is obtained.

3. Pin lewis (Fig. 5.31 *c*)

A pair of strong iron pins are inserted into inclined holes which slope towards each other. The other ends of the pins are connected to chain or rope which in turn is connecting to hoisting arrangement. When the stone is being lifted, the pins tighten against the stone surfaces and a firm grip is obtained.



(a) Chain Dog (b) Chain Lewis



(c) Pin Lewis

FIG. 5.31. CHAIN AND PIN LEWIS.

4. Three legged lewis.

A three legged lewis consists of two dovetailed or tapered steel pieces, a straight steel piece, a U-shaped shakle, a cotter and a round pin. When the three pieces are fixed into the dovetailed hole made at the top of the stone, they are held in position with the help of shakle, cotter and the pin.

Fig. 5.32 shows the stages of assembling three legged lewis. A dovetailed hole is made first. The dovetailed steel pieces are then inserted in the hole. A rectangular steel piece is then inserted or driven between them. The shakle is secured to the three steel pieces by means of pin.

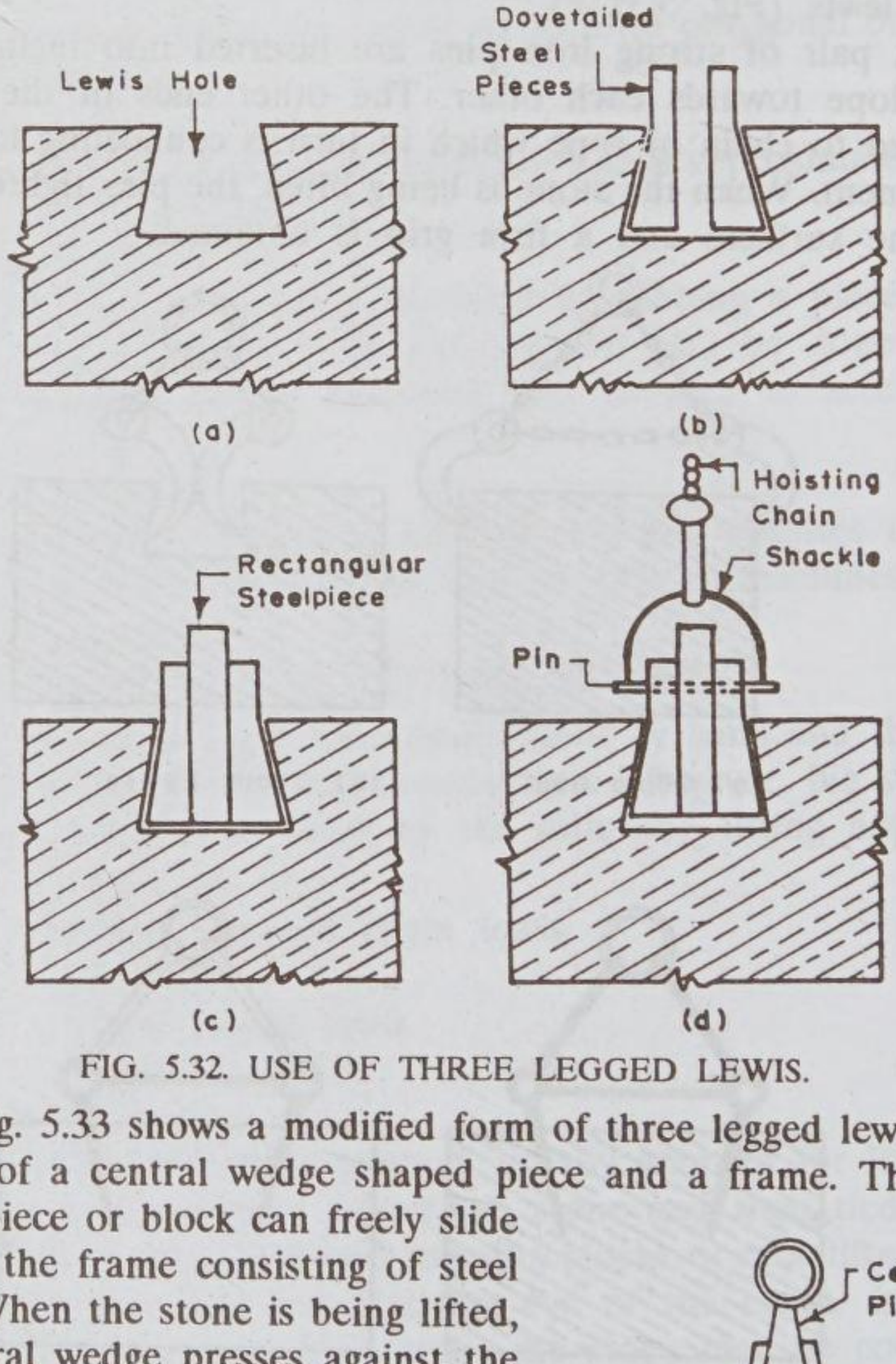


FIG. 5.32. USE OF THREE LEGGED LEWIS.

Fig. 5.33 shows a modified form of three legged lewis, which consists of a central wedge shaped piece and a frame. The wedge shaped piece or block can freely slide between the frame consisting of steel plates. When the stone is being lifted, the central wedge presses against the two sides of the frame.

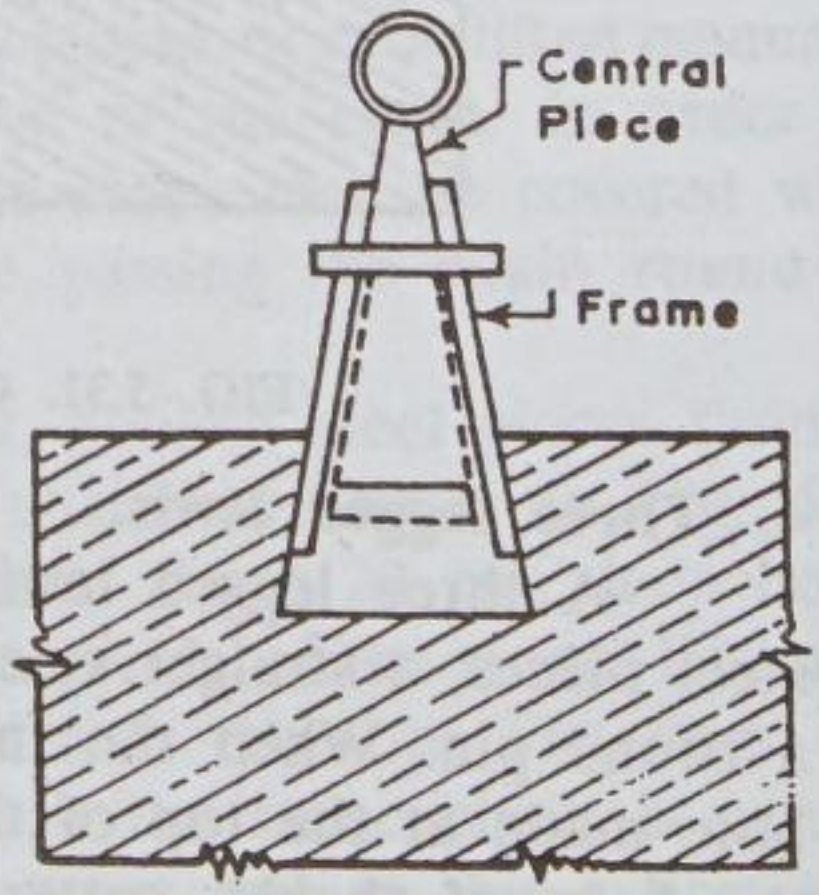


FIG. 5.33. MODIFIED THREE LEGGED LEWIS.

Fig. 5.34 shows a lewis for use under water. It consists of a wedge shaped piece (called plug) having one side vertical and another side inclined, and a straight key with a separate thin chain. The wedge is inserted first and the key is placed next. While lowering the stone in water, the chain of the key is kept slack. When the stone oc-

cupies the position, the thin chain is pulled and the key is taken out.

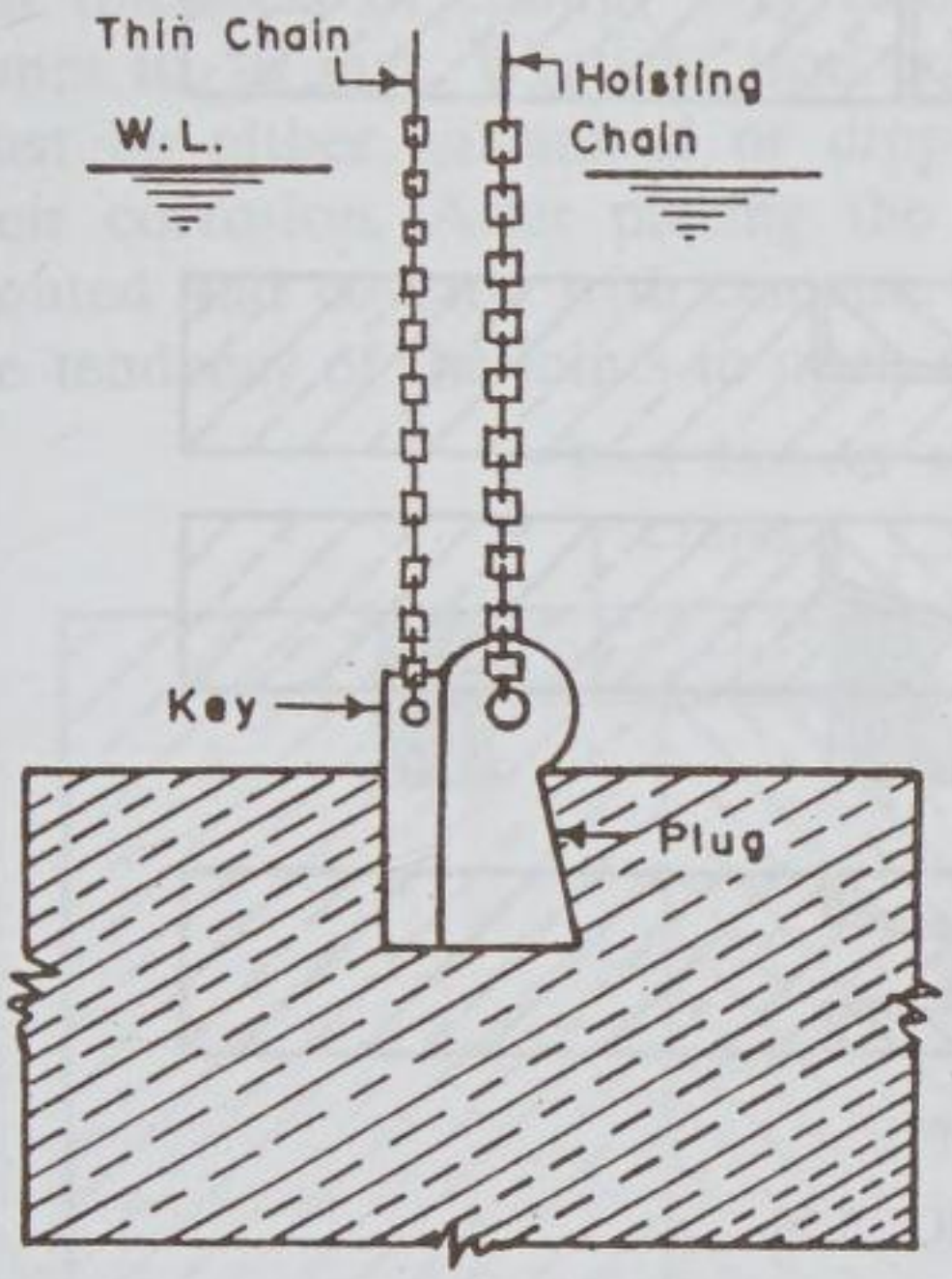


FIG. 5.34. LEWIS FOR LOWERING STONE UNDER WATER.

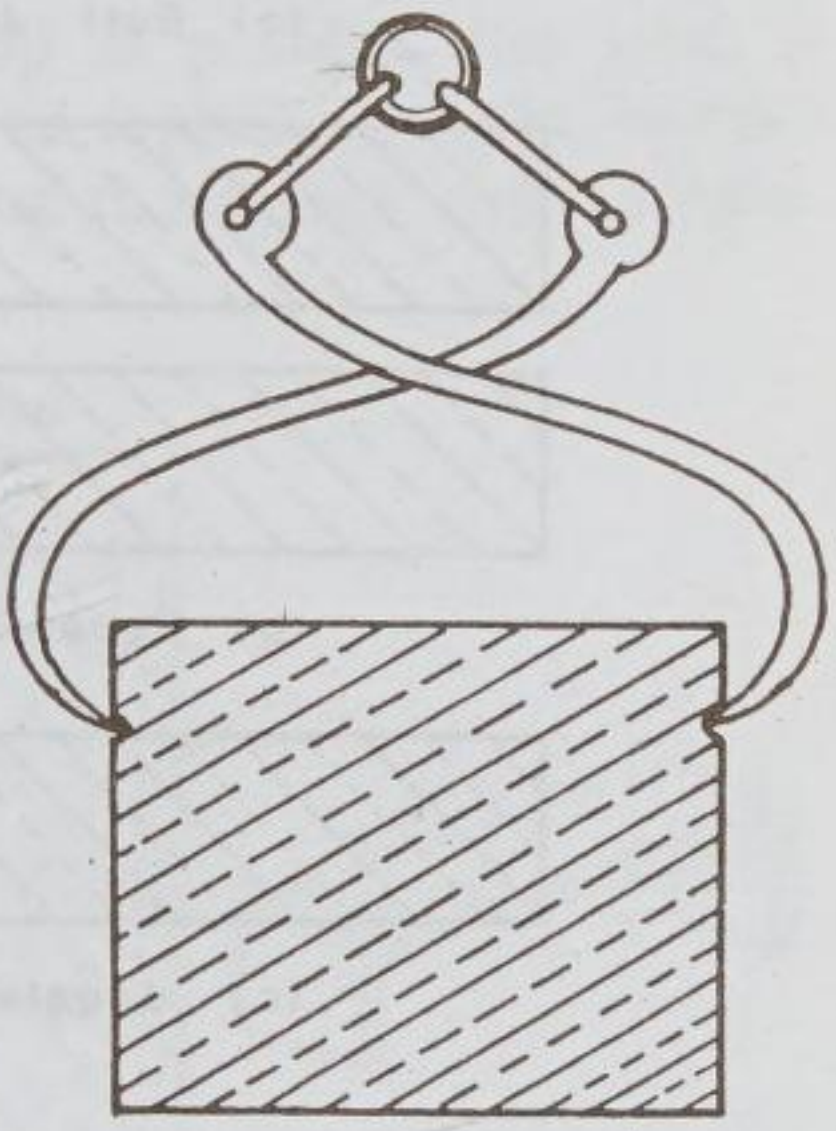


FIG. 5.35. NIPPERS.

5. Nippers

Fig. 5.35 shows the nippers for lifting stones. The pointed ends of the nippers are inserted in the holes made in the sides of the stone, at some distance lower than the top. The hoisting arrangement is connected to the central ring of nipper assembly.

5.7. JOINTS IN STONE MASONRY

Following are the common types of joints provided in stone masonry, to secure the stones firmly with each other :

1. Butt joint or square joint
2. Rebated joint or lapped joint
3. Tongued and grooved joint or joggle joint
4. Bed joint or tabled joint.
5. Cramp joint
6. Plugged joint
7. Dowel joint
8. Rusticated joint

1. Butt joint or square joint

This is most commonly used joint in stone masonry. The dressed edges of two adjacent stones are placed side by side (Fig. 5.36 a).

2. Rebated or lapped joint

This type of joint is provided in arches, gables, copings etc. to prevent the possible movement of the stones. The length of the

rebate or lap depends upon the nature of the work, but it should not be less than 70 mm.

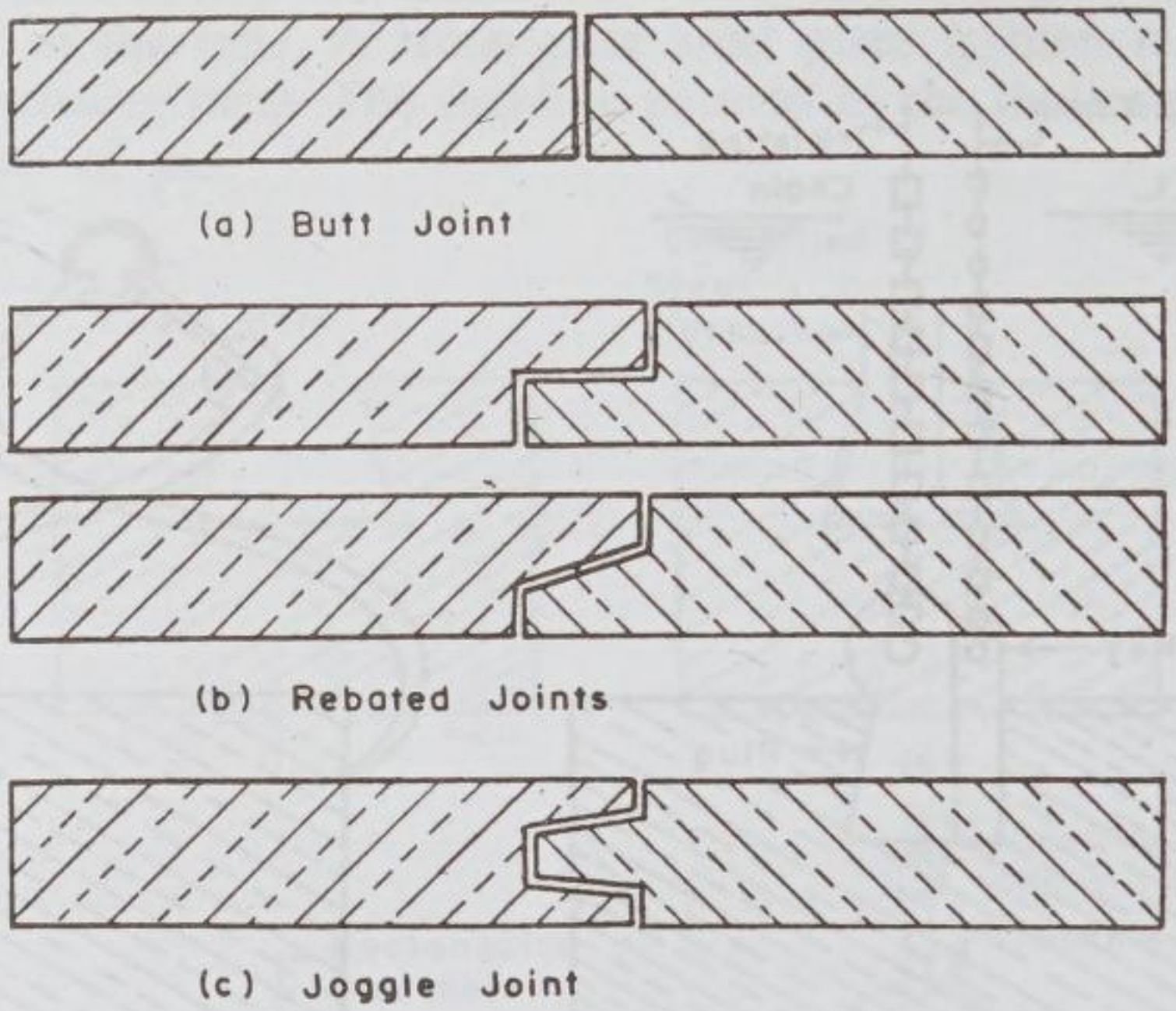


FIG. 5.36.

3. Tongue and grooved joint or joggle joint

This type of joint is provided to prevent sliding along the side joints. The joint is made by providing projection or tongue in one stone and a corresponding groove or sinking on the adjacent stone, as shown in Fig. 5.36 (*c*).

4. Tabled or bed joint

This joint is used to prevent lateral movement of stones such as in sea walls where the lateral pressure is heavy. The joint is made by forming a joggle in the bed of the stone (Fig. 5.37). The height of the projection is kept about 30 to 40 mm, while the width is kept equal to above $\frac{1}{3}$ the breadth of the stone.

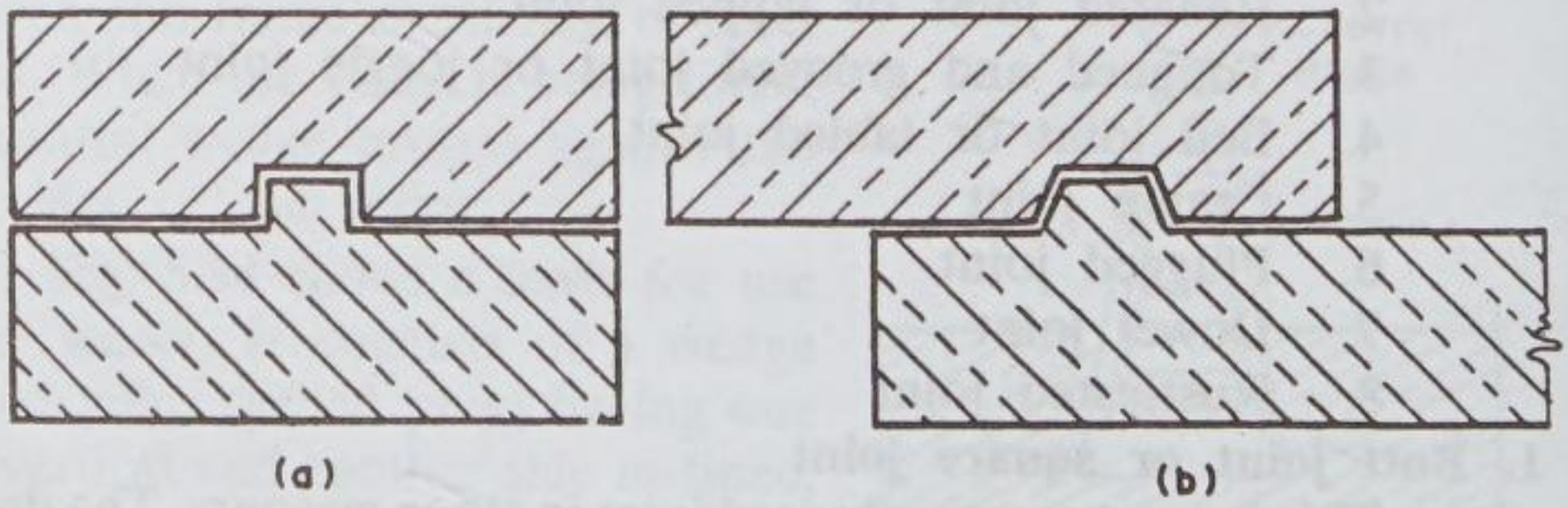


FIG. 5.37. BED JOINT OR TABLED JOINT.

5. Cramped joint (Fig. 5.38)

The joint uses metal cramp instead of dowels. Holes made

in the adjacent stones should be of dovetail shape. The cramps are usually of non-corrosive metals such as gunmetal, copper etc., with their ends turned down to a depth of 4 to 5 cm. The length, width and thickness of cramps vary from 20 to 30 cm, 2 to 4 cm and 5 mm to 10 mm. Wrought iron cramps may also be used but they must be either galvanised or dipped in oil while hot, to prevent their corrosion. After placing the cramp in position, the joint is grouted and covered with cement, lead or asphalt. Cramps prevent the tendency of the joints to open out due to slippage of the stones.

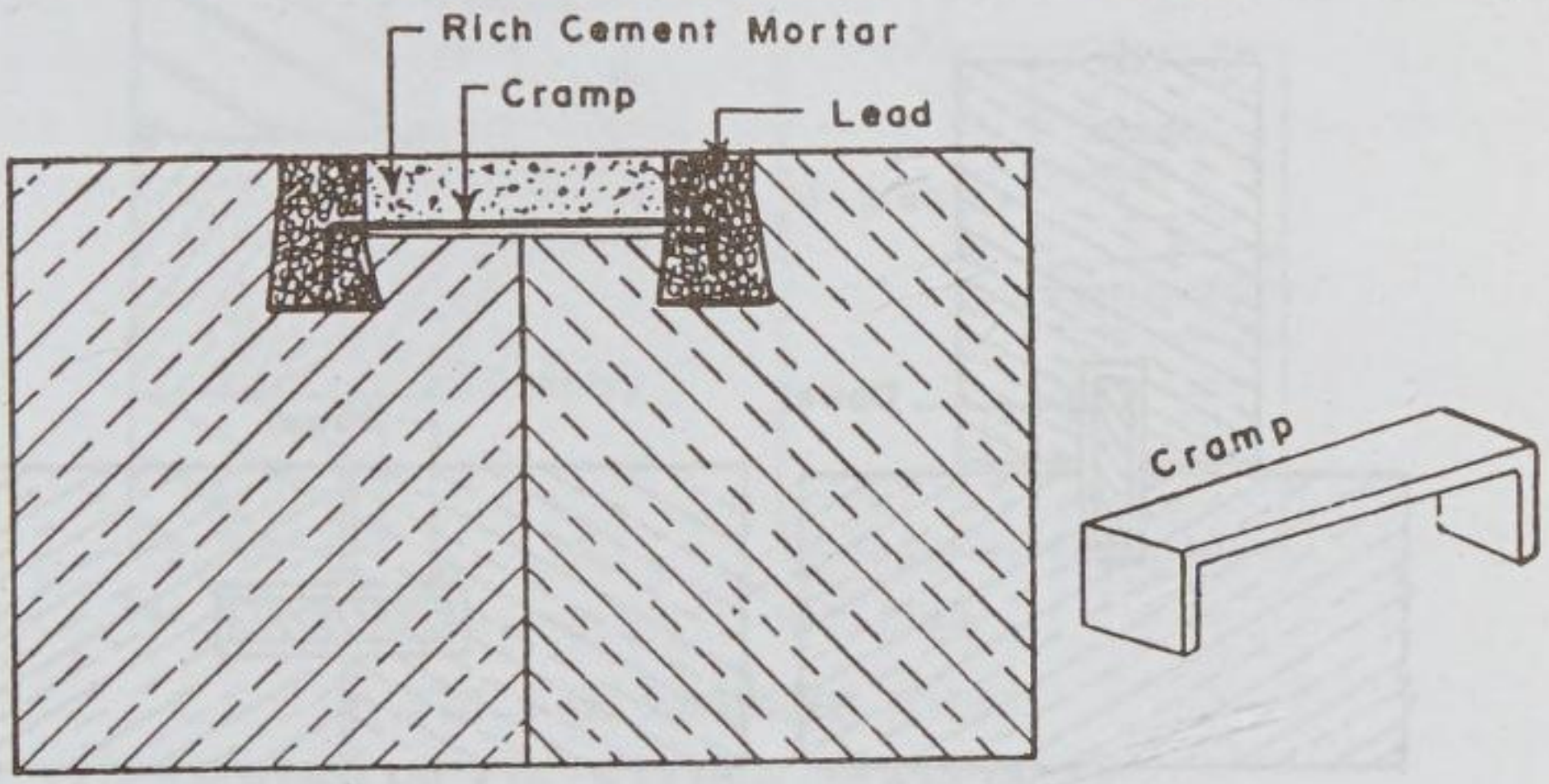


FIG. 5.38. CRAMPED JOINT.

6. Plugged joint

This is an alternative to cramped joints. It consists of making plug holes of dovetail shape in the sides of adjacent stones. After

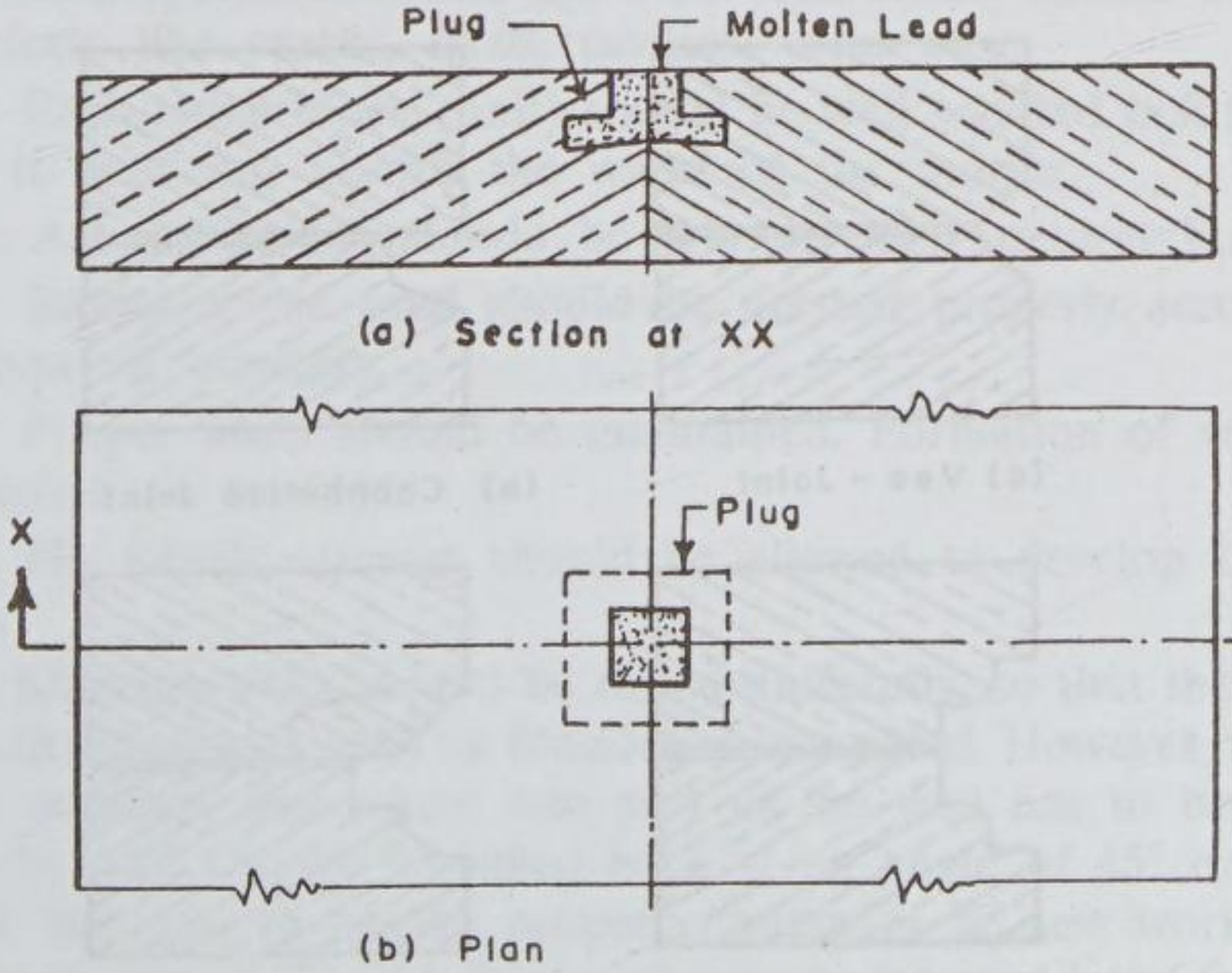


FIG. 5.39. PLUGGED JOINT.

placing the adjacent stones, a common space for plug is formed which is filled with molten lead. Sometimes, rich cement grout is used in the place of molten lead.

7. **Dowelled Joint** (Fig. 5.40)

This is a simple type of joint used to ensure stability of the adjacent stones against displacement or sliding. The joint is formed by cutting rectangular holes in each stone and inserting dowels of hard stone, slate, gunmetal, brass, bronze or copper. These dowels are set in cement mortar.

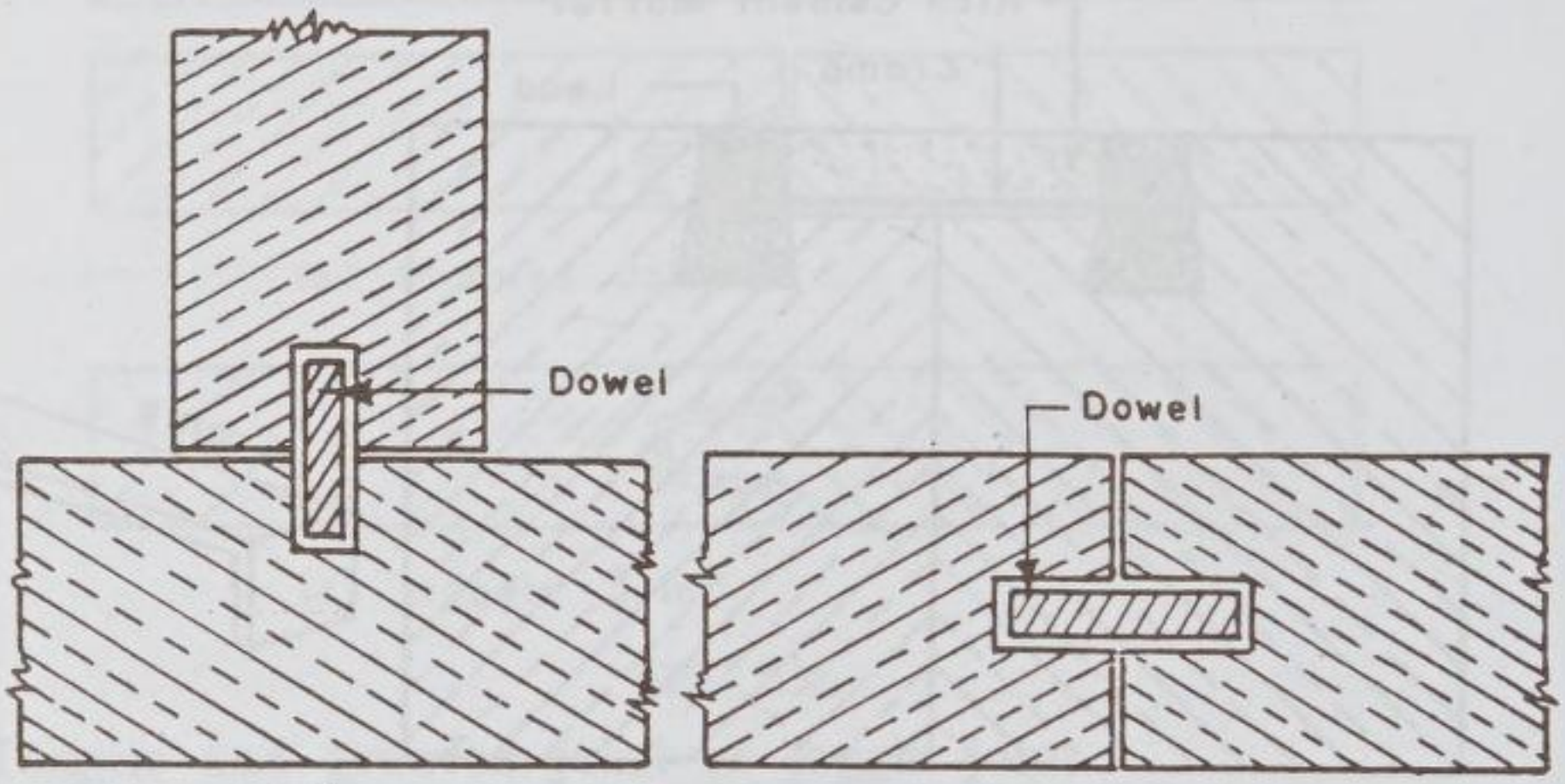


FIG. 5.40. DOWELLED JOINT.

8. Rusticated joint.

This joint is used in those stones whose edges are sunk below the general level, such as for plinth, quoin, outerwalls of lower storeys

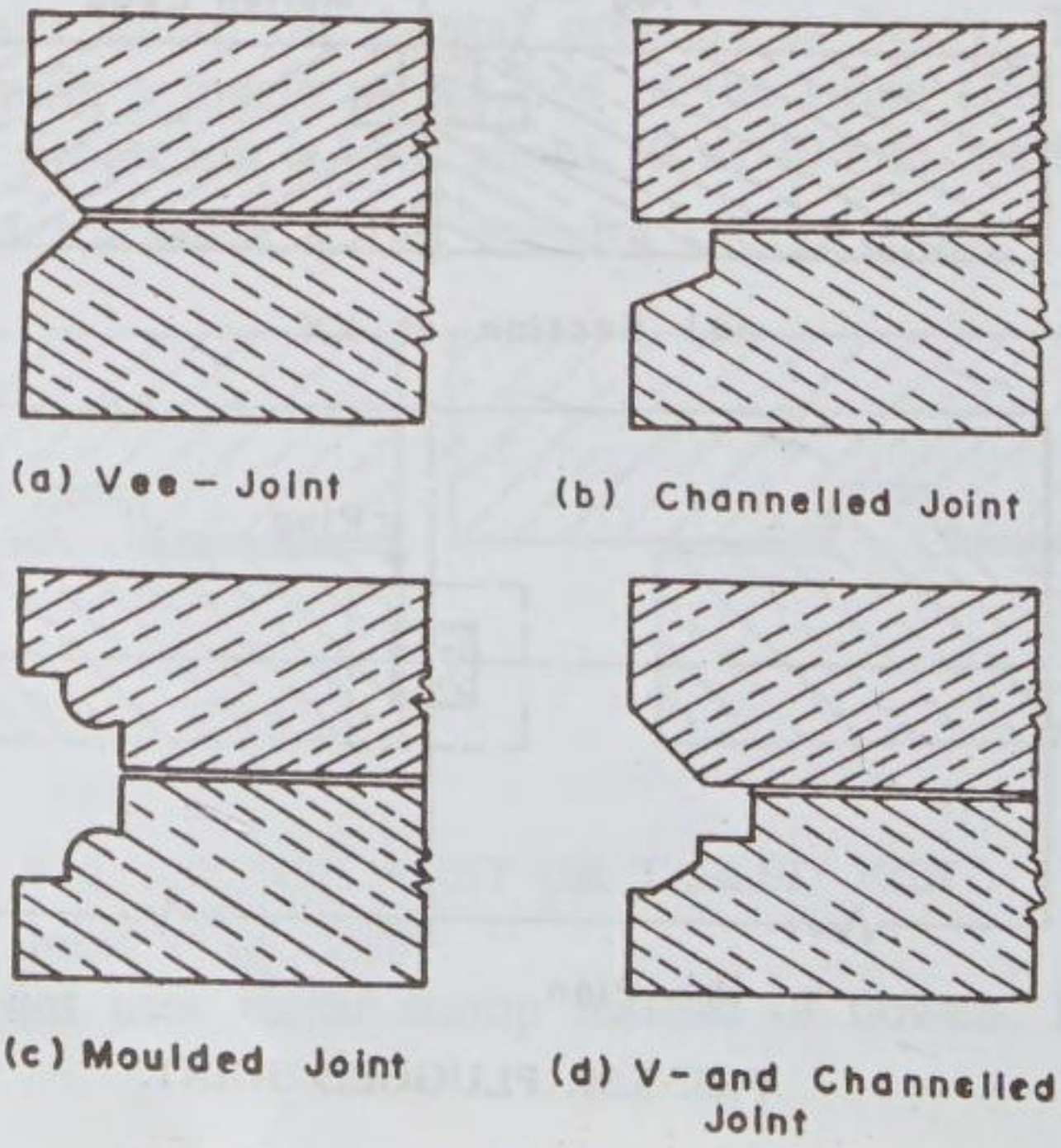


FIG. 5.41. RUSTICATED JOINTS.

etc. Such a joint gives massive appearance to the structure. Various forms of rusticated joints are shown in Fig. 5.41.

9. Saddled or water joint

Such joint is used in cornices and such other weathered surfaces, to divert the water moving on the weathered surface away from the joint. The saddle is bevelled backwards from the front edge, as shown in Fig. 5.42.

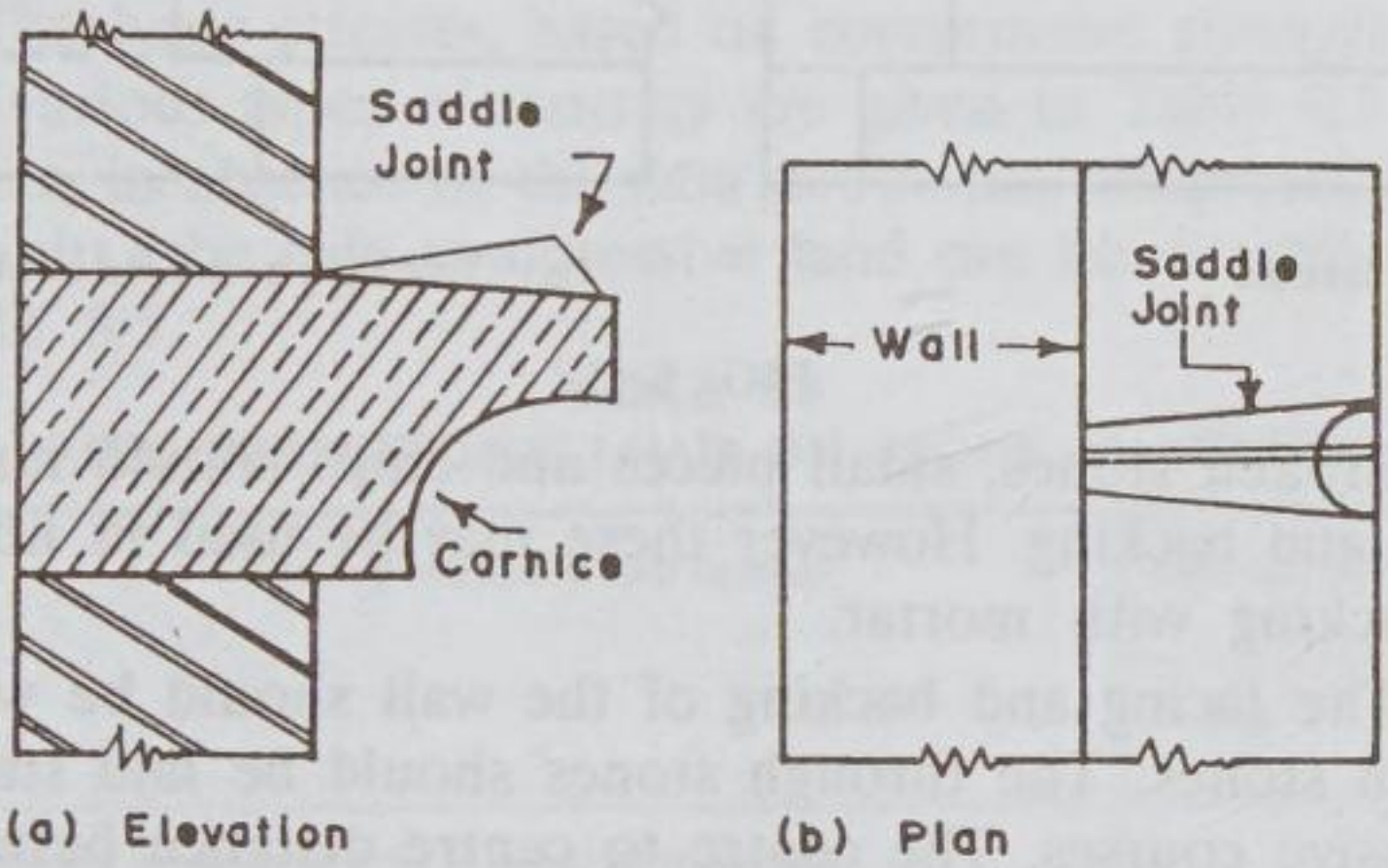


FIG. 5.42. SADDLED JOINT.

5.8. SUPERVISION OF STONE MASONRY CONSTRUCTION

The following points should be kept in mind while supervising the stone masonry work :

1. The stones used should be strong, tough, hard, and should conform with specifications of the work. The stones should be free from defects like cracks, flaws, cavities, veins etc.

2. Each stone block (unit) should be well watered before use so that it does not absorb the water of the mortar.

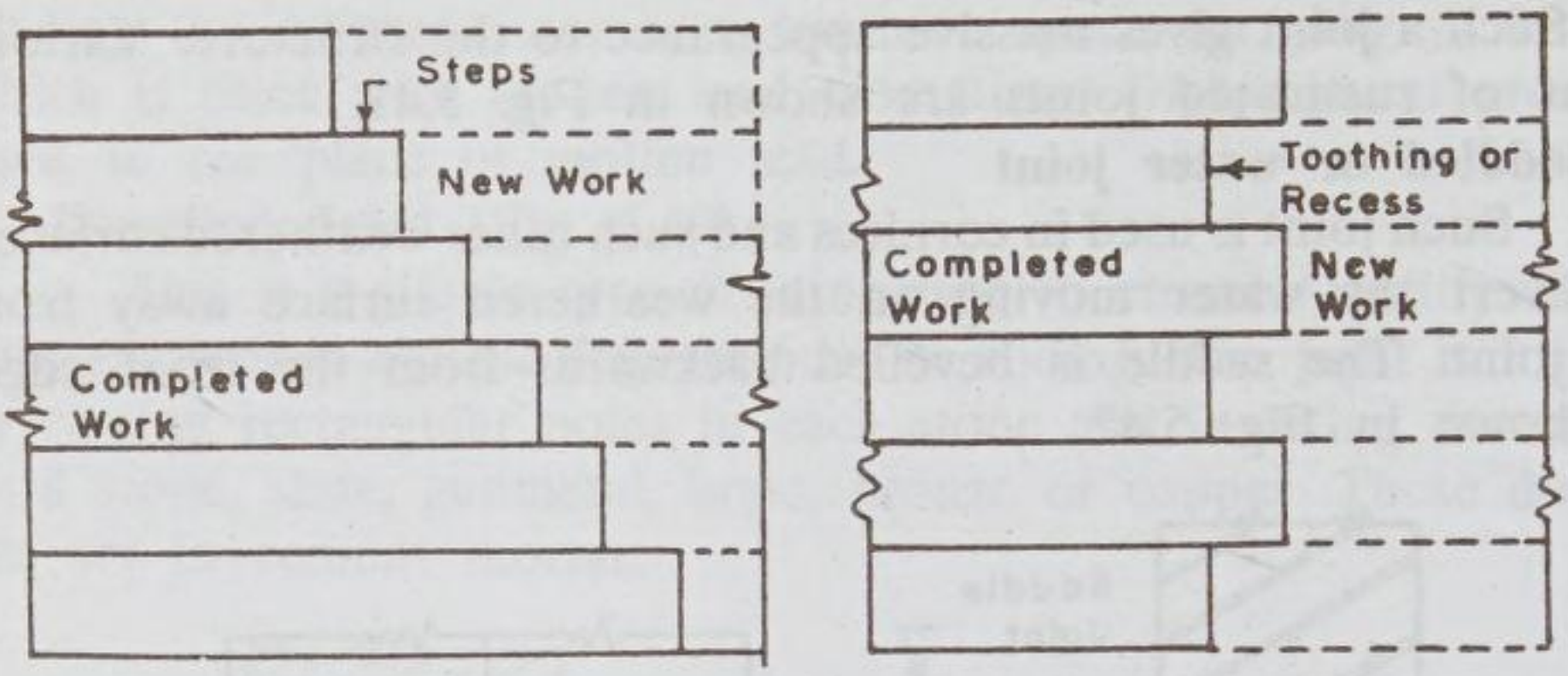
3. All the stones should be laid on their natural bed.

4. Stones to be used should be dressed properly according to the type of masonry.

5. Proper bond should be maintained. Formation of vertical joints should be avoided.

6. No tensile stresses should be allowed to develop in the masonry.

7. Masonry work should be raised uniformly, so that the non-uniform distribution of load on foundation is avoided. However, where it is not possible, and where one part of the wall has to be kept behind, the wall should be raked back at an angle of 45° or less. This will facilitate in having proper connection in new work and the old work (Fig. 5.43). If a cross-wall is to be inserted later, footing or recesses should be provided (Fig. 5.43 *b*).



(a) Steps (b) Toothing or Recess

FIG. 5.43.

8. Broken stones, small pieces and chips should not be used for facing and backing. However these may be used in hearting for proper packing with mortar.

9. The facing and backing of the wall should be well bound by through stones. The through stones should be laid staggered in the successive courses. The centre to centre distance between them should not exceed 1.5 m.

10. The mortar to be used for the work should be of proper quality and proportion. Generally, lime mortar may be used for work above plinth level. But in damp proof construction, cement mortar may be used for masonry below plinth level. Cement mortar may be used for masonry above plinth level also if high compressive strength is required.

11. Quoins used to form the jambs for windows, doors and other openings should be of the full height of the course. The breadth and length of quoin should alteast be 1.5 times and twice its depth respectively.

12. Vertical surfaces (*i.e.* facing and backing) of the wall should be constructed perfectly in plumb. They should be frequently checked.

13. Battered surfaces, if any, should be properly checked with the help of wooden template and plumb.

14. When it is required to raise new construction over the old or dry one, it should be well cleaned and wetted before starting the construction.

15. Double scaffolding should be adopted to carry out the stone masonry construction at higer level.

16. The exposed joints of the masonry should be properly pointed by cement mortar or lime mortar, by raking them first upto a depth of about 2 cm.

17. After the construction is over, the whole work should be cured atleast for 2 to 3 weeks.

5.9. SAFE PERMISSIBLE LOADS ON STONE MASONRY

The strength of stone masonry depends upon the following three factors:

1. Type of stone (*i.e.* sand stone, lime stone etc.)
2. Type of masonry (*i.e.* coursed rubble, Ashlar etc.) and
3. Type of mortar (i.e. lime mortar, cement mortar or gouged mortar). The basic stresses, based on compressive strength of stone units and various types of mortars are given in Table 6.5 (Chapter 6). However, in absence of the data about the compressive strength of stone units, the safe compressive laod can be roughly estimated from Table 5.5

TABLE 5.5
SAFE PERMISSIBLE LOAD ON STONE MASONRY

S.No.	*Type of masonry and mortar*	*safe compressive stress*	
		t/m^2	kN/m^2
1.	Ashlar masonry in 1 : 3 cement mortar		
	(*a*) Granite	160	1600
	(*b*) Sand stone	110	1100
	(*c*) Lime stone	70	700
2.	Ashlar masonry in 1 : 6 cement mortar		
	(*a*) Granite	130	1300
	(*b*) Sand stone	90	900
	(*c*) Lime stone	60	600
3.	Ashlar masonry in lime mortar		
	(*a*) Granite	110	1100
	(*b*) Sand stone	80	800
	(*c*) Lime stone	50	500
4.	Coursed rubble masonry in 1 : 3 cement mortar		
	(*a*) Granite	120	1200
	(*b*) Sand stone	100	1000
	(*c*) Lime stone	60	600
5.	Coursed rubble masonry in 1 : 6 cement mortar		
	(*a*) Granite	100	1000
	(*b*) Sand stone	80	800
	(*c*) Lime stone	50	500
6.	Coursed rubble masonry in lime mortar		
	(*a*) Granite	90	900
	(*b*) Sand stone	70	700
	(*c*) Lime stone	40	400
7.	Random rubble masonry in 1 : 3 cement morter	60—100	600—1000
8.	Random rubble masonry in 1 : 6 cement morter	50—80	500—800
9.	Random rubble masonry in lime morter	40—60-	400—600

PROBLEMS

1. Discuss the comparative merits of various types of building units used for masonry.

2. Define the following terms :

Header ; stretcher ; bond ; quoin ; hearting ; closer ; perpend ; string course ; lacing course ; through stone and reveal.

3. Draw typical sketches, showing the following, constructed of stone masonry.

(*i*) Reveals
(*ii*) Corbel
(*iii*) Cornice
(*iv*) Jambs
(*v*) Coping
(*vi*) Threshold

4. (*a*) Distinguish between (*i*) wall (*ii*) pier (*iii*) buttress and (*iv*) column

(*b*) Draw a typical vertical section of a wall, through openings, and show various important elements of construction.

5. Explain in brief various types of mortars used in stone masonry.

6. What are the requirements of a good mortar ? How do you determine the consistency of mortar ?

7. Classify various types of stone masonry. Draw typical sketches to illustrate the same.

8. Write short notes on the following:
(*i*) Flint walling
(*ii*) Polygonal rubble masonry
(*iii*) Square rubble uncoursed masonry
(*iv*) Random rubble built to courses.

9. Enumerate various types of surface finishes used in stone masonry.

10. Write a note on various appliances used for lifting stones.

11. Explain, with sketches, various types of joints used in ashlar stone masonry.

6

Masonry-2 : Brick Masonry

6.1. INTRODUCTION

Brick masonry is made of brick units bonded together with mortar. Two essential components of brick masonry are therefore :

(*i*) Bricks
(*ii*) Mortar

The mortar used for brick masonry should have the same characteristics as discussed in Chapter 5 for stone masonry. Mortar acts as a cementing material and unites the individual brick units together to act as a homogeneous mass. Following types of mortar may be used in brick masonry :

1. Cement mortar
2. Lime mortar
3. Cement-lime mortar
4. Lime-surkhi mortar and
5. Mud mortar

Mud mortar is used only for low-rise buildings which carry light loads. Cement mortars are used for high-rise buildings, where strength is of prime importance. Lime mortar and lime-surkhi mortars are used for all types of construction.

Bricks are manufactured by moulding clay in rectangular blocks of uniform predetermined size, drying them and then burning them in a kiln. Clay is a plastic earth, constituted largely of sand and alumina with traces of chalk, iron, manganese dioxide etc. Good bricks should be thoroughly burnt so that they become hard and durable. Satisfactory burning of bricks is ascertained by a hard ringing sound emitted when two bricks are struck together. The bricks should be free from cracks, chips, and large particles of lime.

The strength of brick masonry chiefly depends upon: (*i*) quality

of bricks, (*ii*) quality of mortar and (*iii*) method of bonding used. Unbonded wall, even constructed with good quality bricks and good quality mortar has little strength and stability.

Brick masonry is sometimes preferred over other types of masonry due to the following reasons :

1. All the bricks are of uniform size and shape, and hence they can be laid in any definite pattern.
2. Brick units are light in weight and small in size. Hence these can be easily handled by brick layers by hand.
3. Bricks do not need any dressing.
4. The art of brick laying can be understood very easily, and even unskilled masons can do the brick masonry. Stone masonry construction requires highly skilled masons.
5. Bricks are easily available at all sites, unlike stones which are available only at quarry sites. Due to this, they do not require transportation from long distances.
6. Ornamental work can be easily done with bricks.
7. Light partition walls and filler walls can be easily constructed in brick masonry.

6.2. TYPES OF BRICKS

Bricks used in masonry can be of two types :

(*i*) Traditional bricks
(*ii*) Modular bricks.

Traditional bricks are those which have not been standardized in size. The dimensions of traditional bricks vary from place to place. Their length varies from 20 to 25 cm, width varies from 10 to 13 cm and thickness varies from 5 cm to 7.5 cm. The commonly adopted nominal size of traditional brick is 23 cm× 11.4 cm× 7.6 cm ($9''\times4\frac{1}{2}''\times3''$) approximately.

Modular bricks conform to the size laid down by Bureau of Indian Standard Institution, India. Any brick which is of the same uniform size as laid down by BIS is known as the *modular brick.* The *nominal size* of the modular brick is 20 cm × 10 cm × 10 cm while the actual size of the brick is 19 cm × 9 cm × 9 cm. *Nominal size* includes the mortar thickness. Masonry modular bricks are economical to manufacture, require less area for drying, and staking, and requires less *brick work* for the same surface area of the wall, in comparison to conventional bricks. The masonry with modular bricks thus workout to be cheaper.

Classes of bricks

Quality wise, masonry bricks are classified into three classes: (*i*) First class bricks (*ii*) Second class bricks and (*iii*) Third class bricks.

(*i*) ***First class bricks.*** First class bricks are those which

strictly conform to the standard size of modular bricks, *i.e.*. 19 cm × 9 cm × 9 cm actual size, such that ten layers of brick laid in mortar will form masonry of 1 metre height. Good bricks are manufactured from good quality plastic earth which is free from saline deposits. They are of good uniform colour. They are well burnt; hard ringing sound is emitted when two bricks are struck together. They have straight edges and even surfaces. They are free from cracks, chips, flaws and nodules of lime. When immersed in water for one hour, they do not absorb water more than one-sixth of their weight, on drying, they do not show any sign of afflorescence.

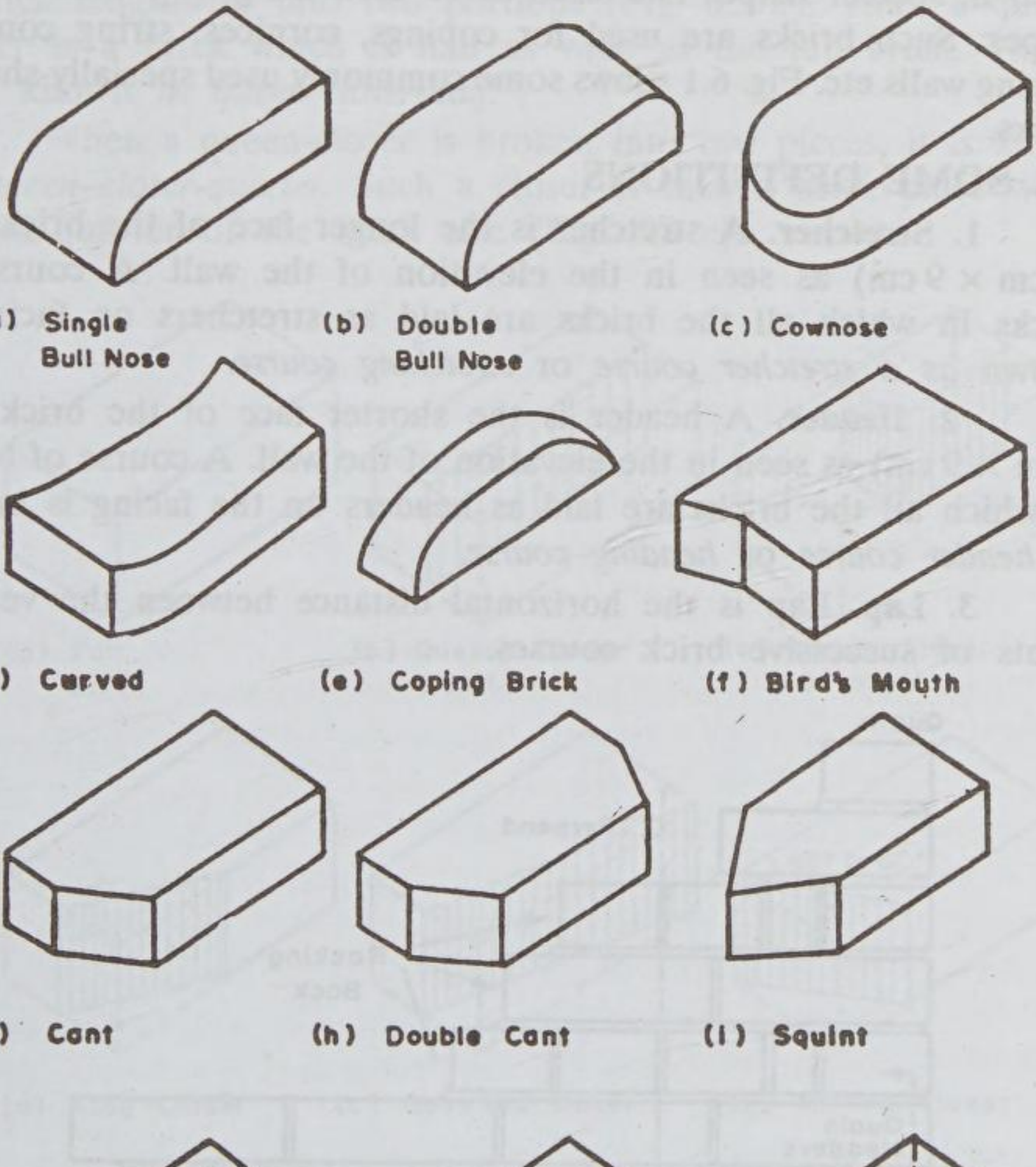


FIG. 6.1. SPECIALLY-SHAPED BRICKS.

(*ii*) ***Second class bricks.*** Second class bricks also conform to the standard size, but they are slightly, irregular in shape and colour. They are also fully burnt, and ringing sound is emitted when two bricks are struck together. When immersed in water for one hour, they do not absorb water more than one-fourth of their weight.

(*iii*) ***Third class bricks.*** These are the one which are quite irregular in their size, shape and finish. They are not burnt fully, due to which they are of reddish-yellow colour. These bricks have low crushing strength. They are not used for quality brick-masonry.

Moulded bricks. Moulded bricks are those which are manufactured in special shapes and sizes to be used for giving architectural shapes. Such bricks are used for copings, cornices, string courses, sloping walls etc. Fig. 6.1 shows some commonly used specially-shaped bricks.

6.3. SOME DEFINITIONS

1. Stretcher. A stretcher is the longer face of the brick (*i.e.* 19 cm × 9 cm) as seen in the elevation of the wall. A course of bricks in which all the bricks are laid as stretchers on facing is known *as a stretcher course* or *stretching course.*

2. Header. A header is the shorter face of the brick (*i.e.* 9 cm × 9 cm) as seen in the elevation of the wall. A course of bricks in which all the bricks are laid as headers on the facing is known as *header course* or *heading course.*

3. Lap. Lap is the horizontal distance between the vertical joints of successive brick courses.

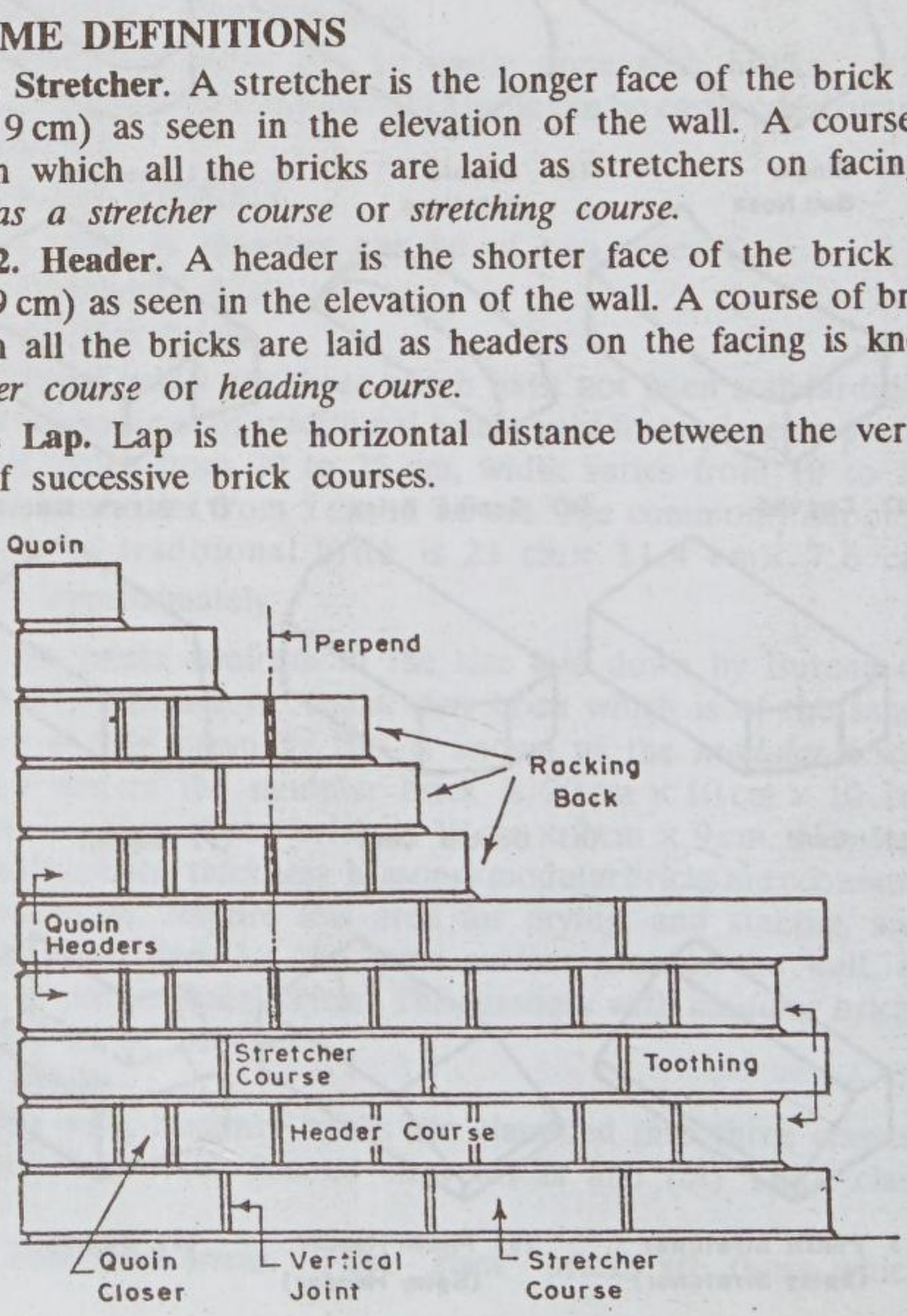


FIG. 6.2. ELEVATION OF A BRICK WALL.

4. Perpend. A perpend is an imaginary vertical line which includes the vertical joint separating two adjoining bricks.

5. Bed. Bed is the lower surface (19 cm × 9 cm) of the brick when laid flat.

6. Closer. It is a portion of a brick with the cut made *longitudinally,* and is used to close up bond at the end of the course. A closer helps in preventing the joints of successive sources (higher or lower) to come in a vertical line. Closers may be of various types, defined below.

7. Queen-closer. It is a portion of a brick obtained by cutting a brick lengthwise into two portions (Fig. 6.3 *b*). Thus, a queen-closer is a brick which is half as wide as the full brick. This is also known as *queen-closer-half.*

When a queen-closer is broken into two pieces, it is known as *queen–closer-quarter.* Such a closer is thus a brick piece which is one-quarter of the brick size (Fig. 6.3 *c*).

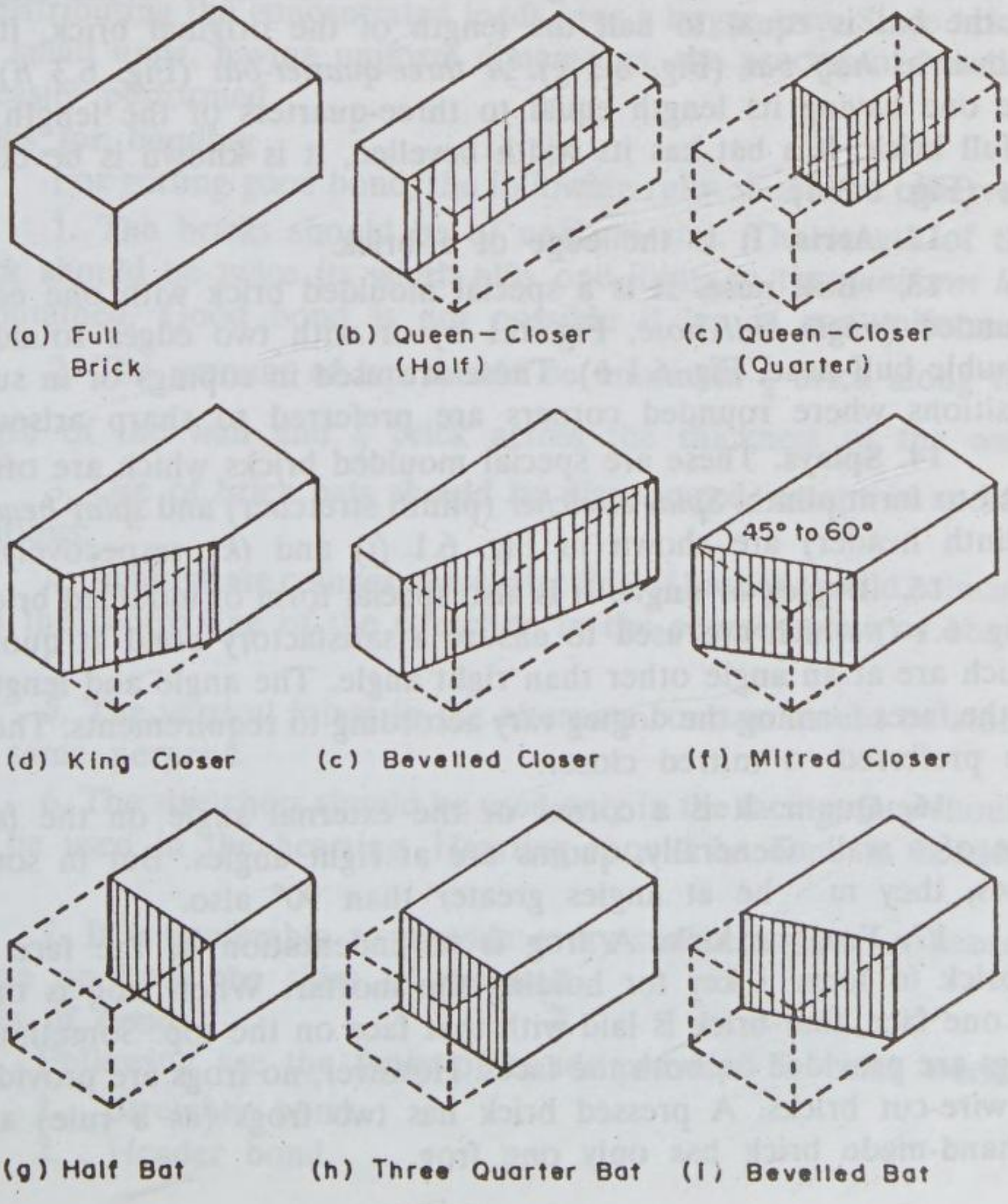


FIG. 6.3. VARIOUS FORMS OF BRICK PORTIONS.

8. King closer. It is the portion of a brick which is so cut that the width of one its end is half that of a full brick, while the width at the other end is equal to the full width (Fig. 6.3 *d*). It is thus obtained by cutting the triangular piece between the centre of one end and the centre of the other (lay) side. It has half-header and half-stretcher face.

9. Bevelled closer. It is a special form of a king closer in which the whole length of the brick (*i.e.*. stretcher face) is bevelled in such a way that half width is maintained at one end and full width is maintained at the other end (Fig. 6.3 *e*).

10. Mitred closer. It is a portion of a brick whose one end is cut splayed or mitred for full width. The angle of splay may vary from 45° to 60°. Thus, one longer face of the mitred closer is of full length of the brick while the other longer face is smaller in length (Fig. 6.3 *f*).

11. Bat. It is the portion of the brick cut across the width. Thus, a bat is smaller in length than the full brick. If the length of the bat is equal to half the length of the original brick, it is known as *half bat* (Fig. 6.3 *g*). *A three-quarter-bat* (Fig. 6.3 *h*) is the one having its length equal to three-quarters of the length of a full brick. If a bat has its width bevelled, it is known is bevelled bat (Fig. 6.3 *i*).

12. Arris. It is the edge of a brick.

13. Bull nose. It is a special moulded brick with one edge rounded (*single bull nose*, Fig. 6.1 *a*) or with two edges rounded (double bull nose, Fig. 6.1 *b*). These are used in copings or in such positions where rounded corners are preferred to sharp arises.

14. Splays. These are special moulded bricks which are often used to form plinth. *Splay stretcher* (plinth stretcher) and *splay header* (plinth header) are shown in Fig. 6.1 (*j*) and (*k*) respectively.

15. Dogleg or angle. It is also special form of moulded bricks (Fig. 6.1 *l*) which are used to ensure a satisfactory bond at quoins which are at an angle other than right angle. The angle and lengths of the faces forming the dogleg vary according to requirements. These are preferred to mitred closer.

16. Quoin. It is a corner or the external angle on the face side of a wall. Generally, quoins are at right angles. But in some cases, they may be at angles greater than 90° also.

17. Frog or kick. A frog is an indentation in the face of a brick to form a key for holding the mortar. When frog is only on one face, that brick is laid with that face on the top. Sometimes, frogs are provided on both the faces. However, no frogs are provided in wire-cut bricks. A pressed brick has two frogs (as a rule) and a hand-made brick has only one frog.

18. Racking back. It is the termination of a wall in a stepped fashion, as shown in Fig. 6.2.

19. Toothing. It is the termination of the wall in such a fashion that each alternate course at the end projects, in order to provide adequate bond if the wall is continued horizontally at a later stage (Fig. 6.2).

6.4. BONDS IN BRICK WORK

Bond is the interlacement of bricks, formed when they lay (or project beyond) those immediately below or above them. It is the method of arranging the bricks in courses so that individual units are tied together and the vertical joints of the successive courses do not lie in same vertical line. Bond of various types are distinguished by their elevation or face appearance. Bricks used in masonry are all of uniform size. If they are not arranged (or bonded) properly, continuous vertical joints will result. *An unbonded wall*, with its continuous vertical joints has little strength and stability. Bonds help in distributing the concentrated loads over a larger area. Since bricks are small units, having uniform dimensions, the process of bonding is easily performed.

Rules for bonding

For getting good bond, the following rules should be observed:

1. The bricks should be of uniform size. The length of the brick should be twice its width plus one joint, so that *uniform lap* is obtained. Good bond is not possible if lap is non-uniform.

2. The amount of lap should be minimum $\frac{1}{4}$ brick along the length of the wall and $\frac{1}{2}$ brick across the thickness of the wall.

3. Use of brick bats should be discouraged, except in special locations.

4. In alternate courses, the centre line of header should coincide with the centre line of the stretcher, in the course below or above it.

5. The vertical joints in the alternate courses should be along the same perpend.

6. The stretchers should be used only in the facing; they should not be used in the hearting. Hearting should be done in headers only.

7. It is preferable to provide every sixth course as a header course on both the sides of the wall.

Types of bonds.

Following are the types of bonds provided in brick work :

1. Stretcher bond.
2. Header bond.

3. English bond.
4. Flemish bond.
5. Facing bond.
6. English cross bond.
7. Brick on edge bond.
8. Dutch bond.
9. Raking bond.
10. Zigzag bond.
11. Garden wall bond.

6.5. STRETCHER BOND

Stretcher bond or stretching bond is the one in which all the bricks are laid as stretchers on the faces of walls. The length of the bricks are thus along the direction of the wall. This pattern is used only for those walls which have thickness of half brick (*i.e.* 9 cm). Such as those used as partition walls, sleeper walls, division walls or chimney stacks. The bond is not possible if the thickness of the wall is more.

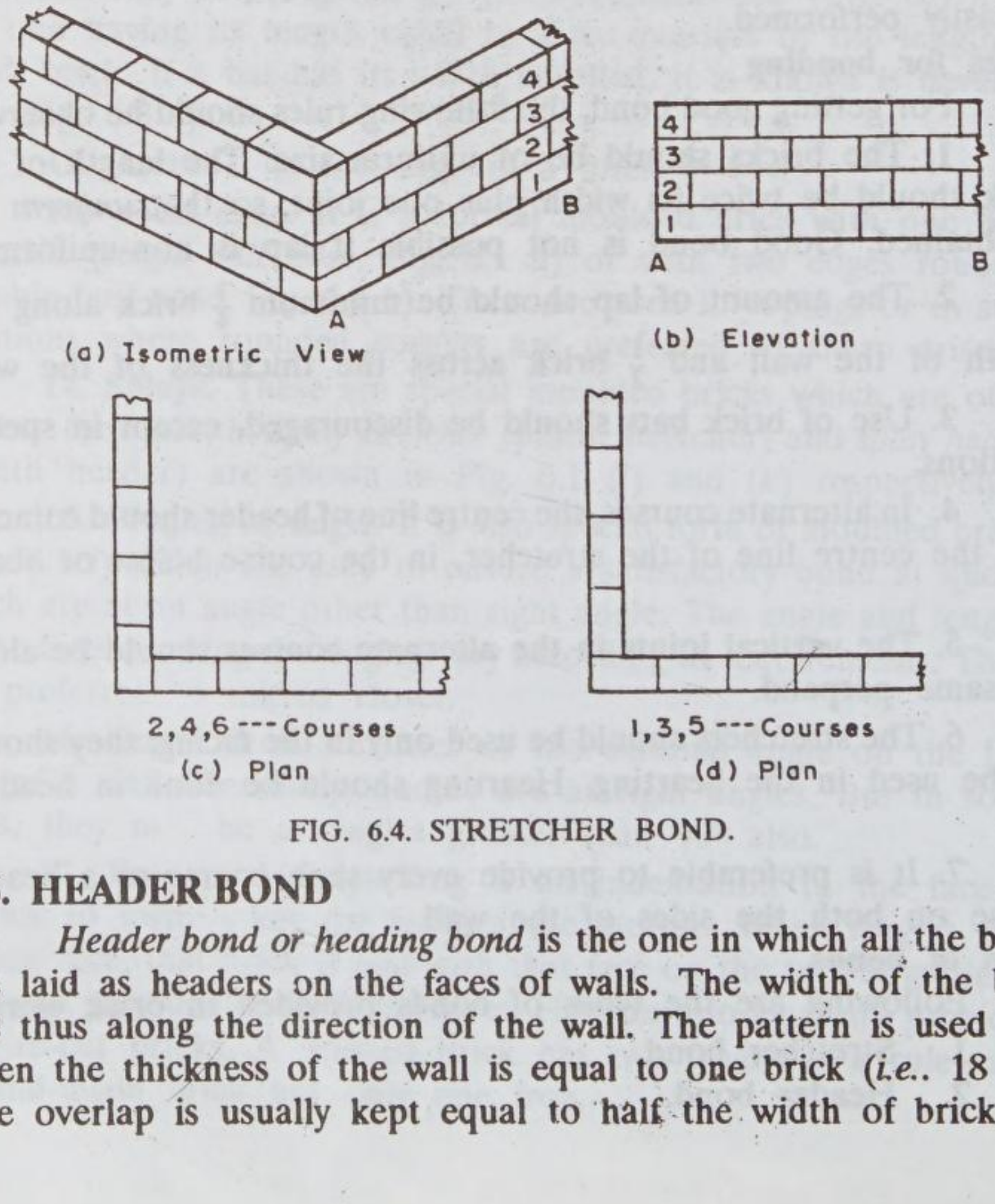


FIG. 6.4. STRETCHER BOND.

6.6. HEADER BOND

Header bond or heading bond is the one in which all the bricks are laid as headers on the faces of walls. The width of the brick are thus along the direction of the wall. The pattern is used only when the thickness of the wall is equal to one brick (*i.e.*. 18 cm). The overlap is usually kept equal to half the width of brick (*i.e.*

$4\frac{1}{2}$ cm). This is achieved by using three-quarter brick bats in each alternate courses as quoins. This bond does not have strength to transmit pressure in the direction of the length of the wall. As such, it is unsuitable for load bearing walls. However, the bond is specially useful for curved brick work where the stretchers, if used, would

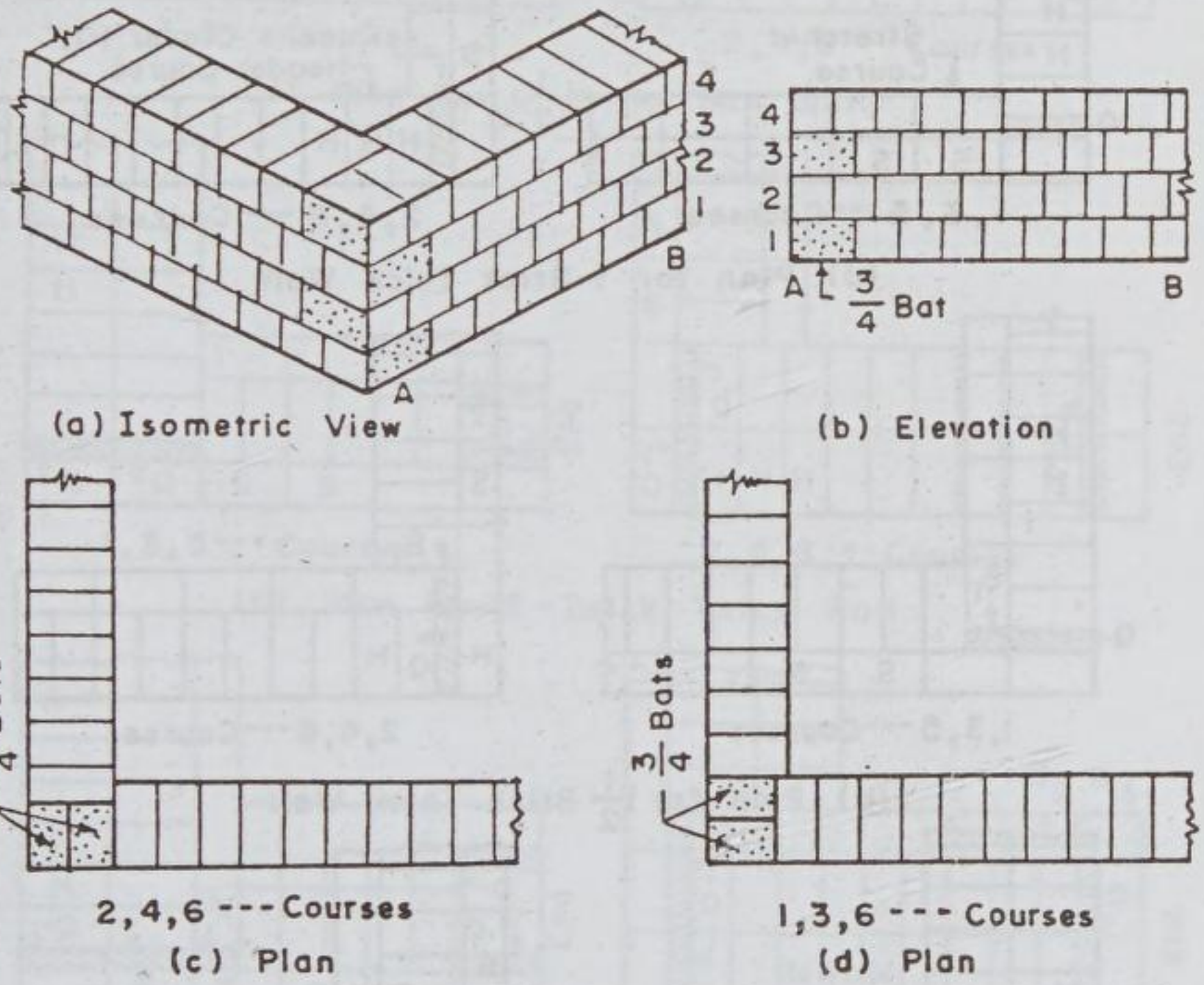
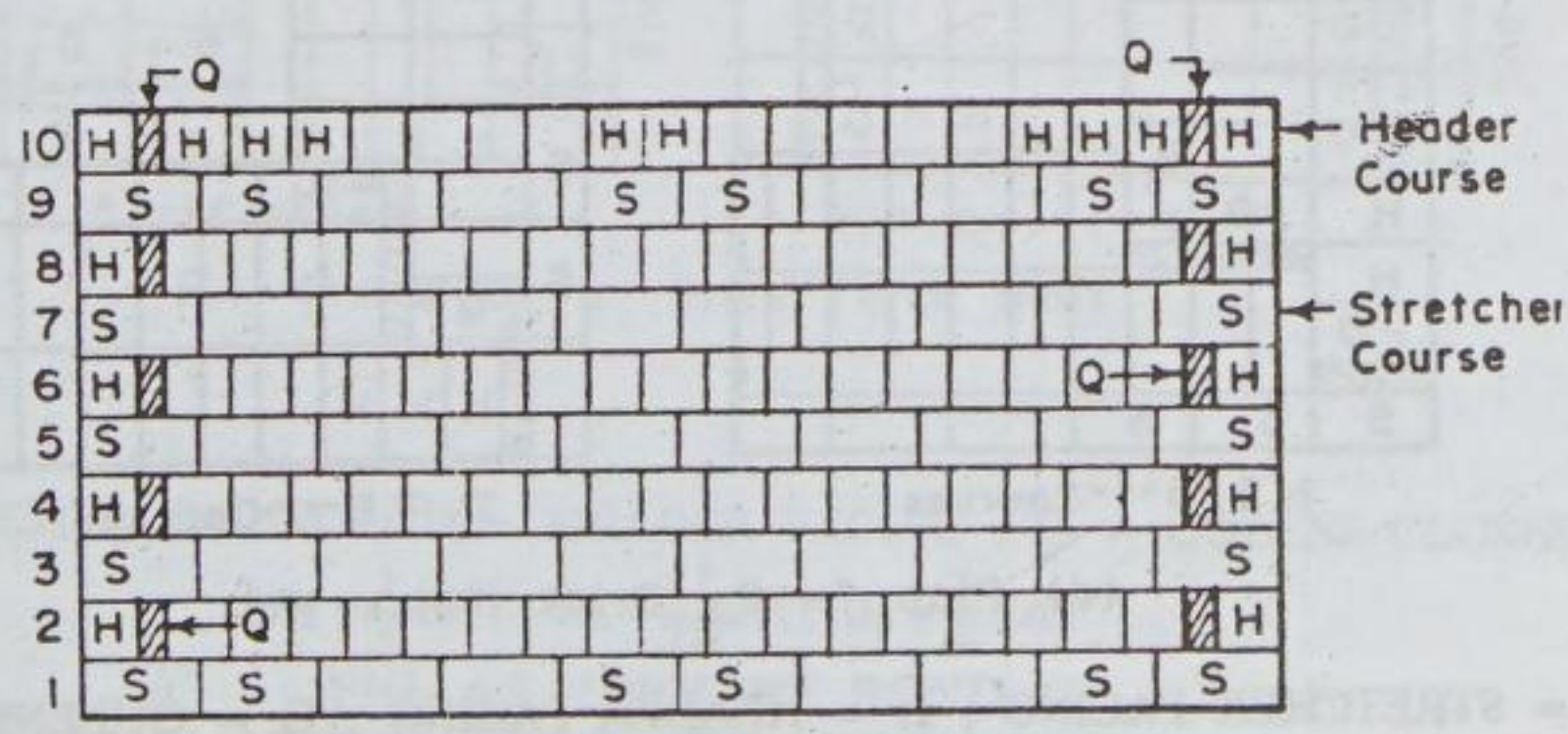


FIG. 6.5. HEADER BOND

project beyond the face of the wall and would necessitate inconvenient cutting. This is also used in construction of footings.

6.7. ENGLISH BOND

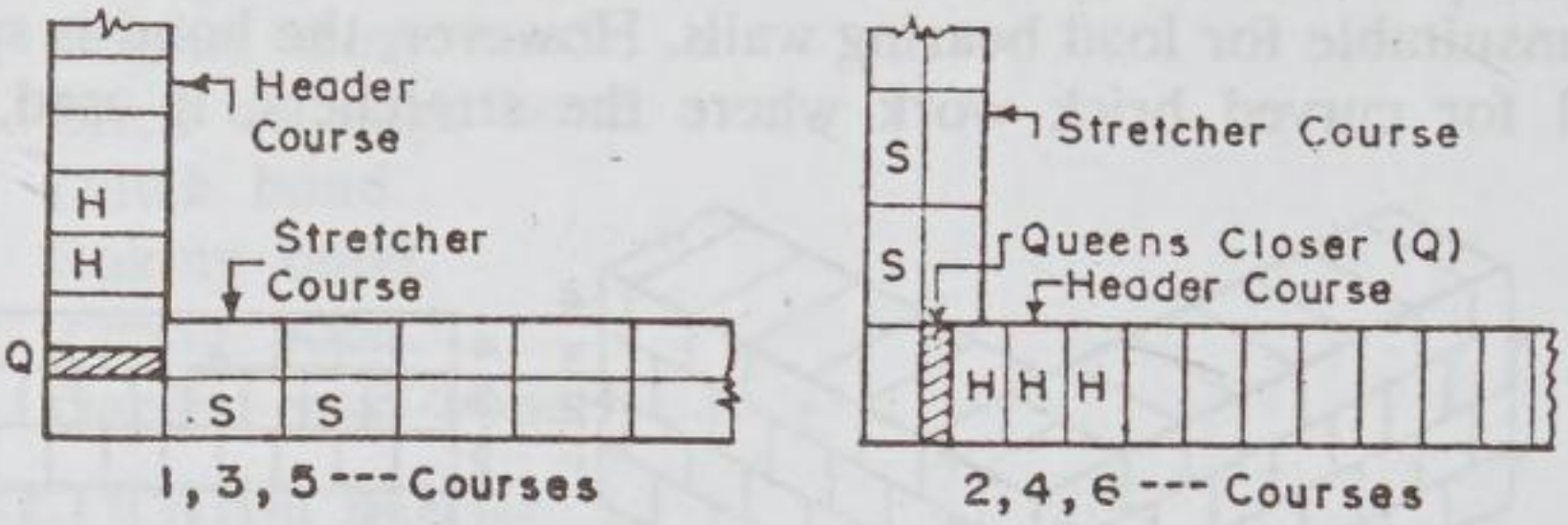
This is the most commonly used bond, for all wall thicknesses. This bond is considered to be the strongest. The bond consists of *alternate courses* of headers and stretchers. In this bond, the vertical joints of the header courses come over each other; similarly, the


Q
Q
10 H S H S H S H S H S
9 S H S H S H S H S H
8 H S H S H S H S H S
7 S H S H S H S H S H
6 H S H S H S H S H S
5 S H S H S H S H S H
4 H S H S H S H S H S
3 S H S H S H S H S H
2 H Q S H S H S H S H
1 S H S H S H S H S H
Header Course
Stretcher Course

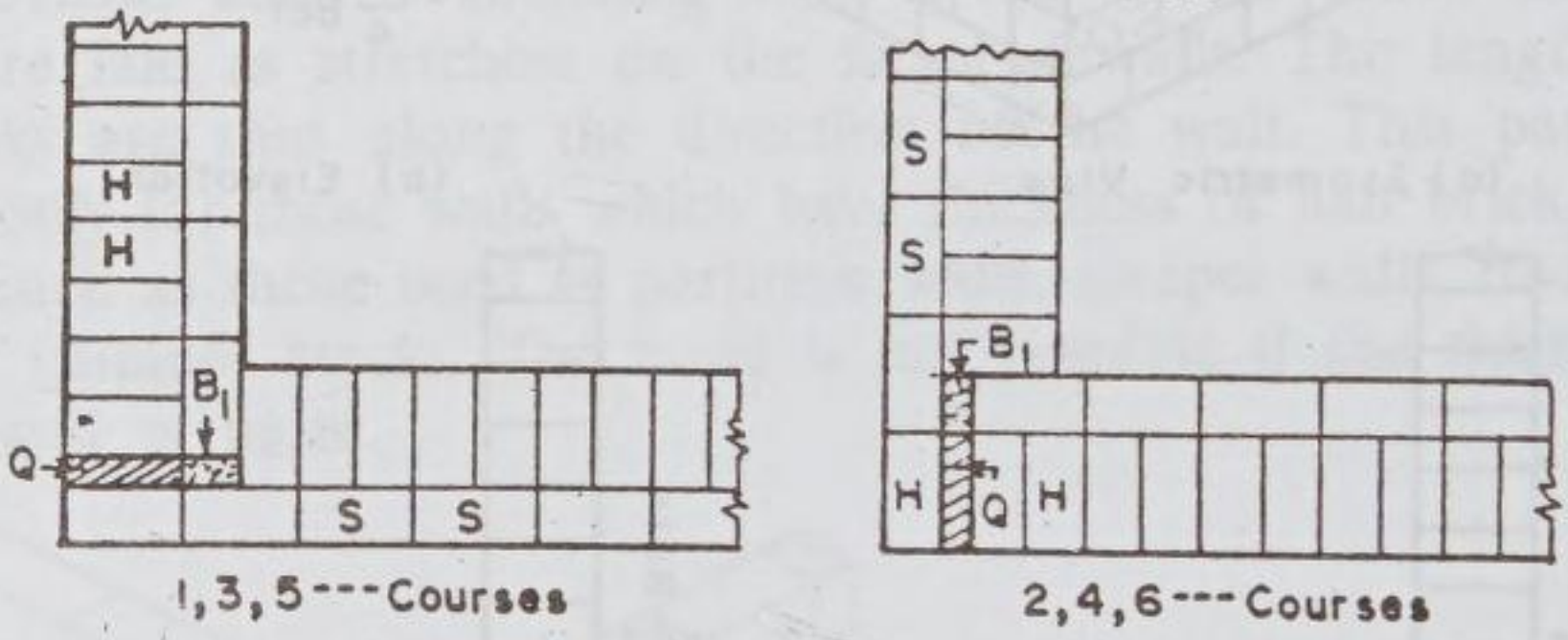
S = STRETCHER ; H = HEADER ; Q = QUEEN CLOSER

FIG. 6.6. ENGLISH BOND.

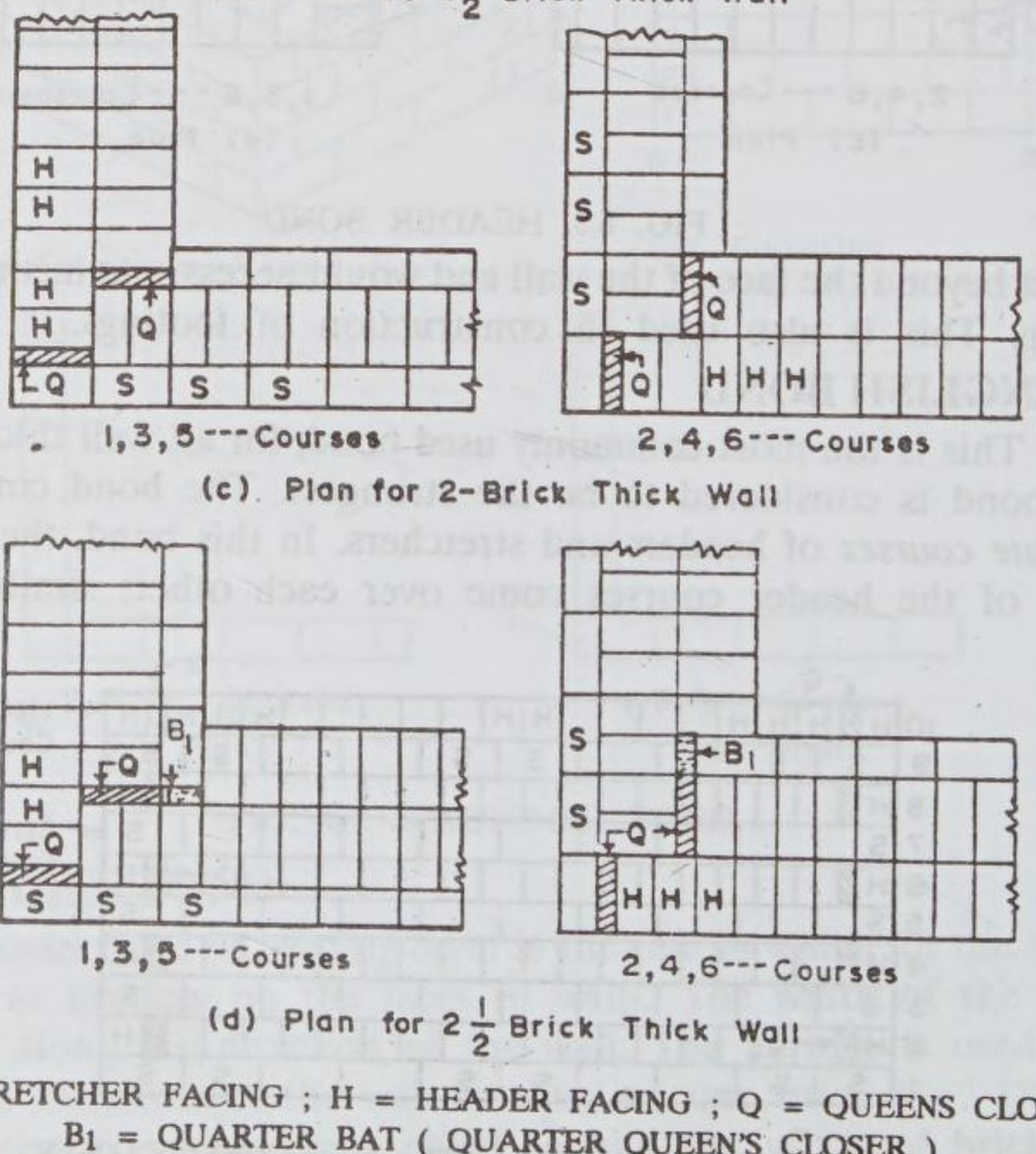
vertical joints of the stretcher courses also come over each other. In order to break the vertical joints in the successive courses, it is essential to place queen closer after the first header (quoin header)



(a) Plan for 1 Brick Thick Wall



(b) Plan for $1\frac{1}{2}$ Brick Thick Wall

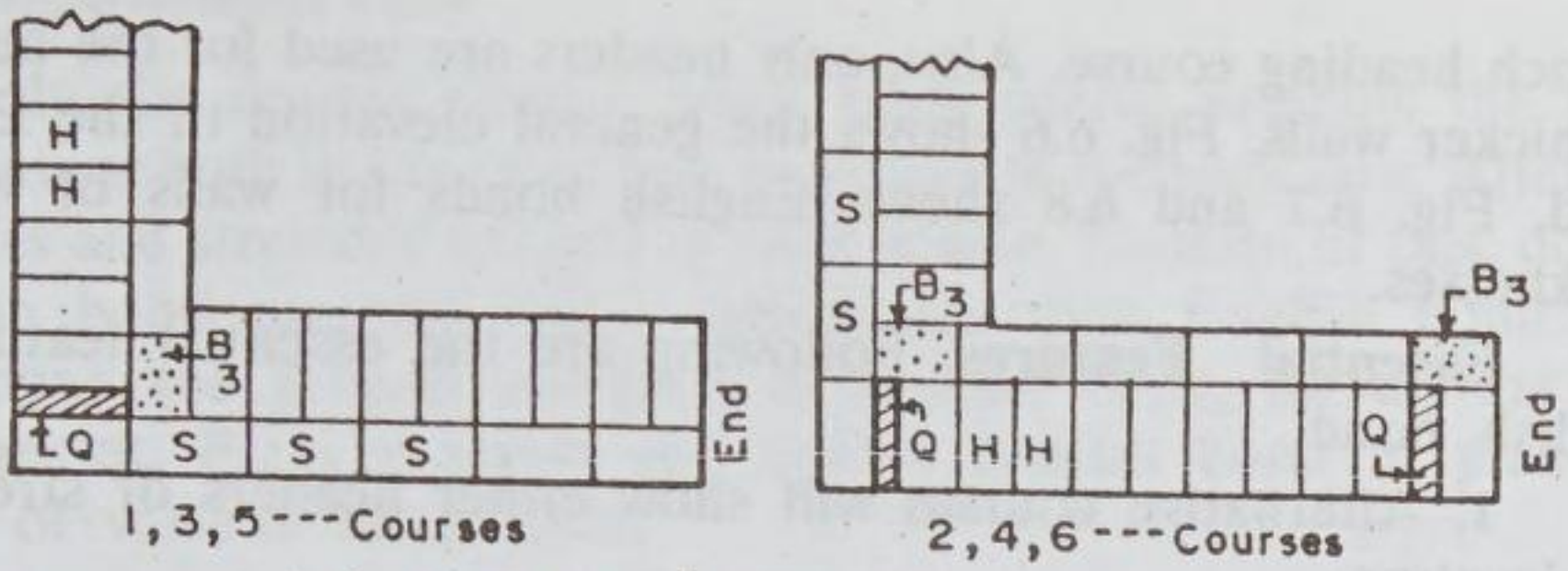


(c) Plan for 2-Brick Thick Wall

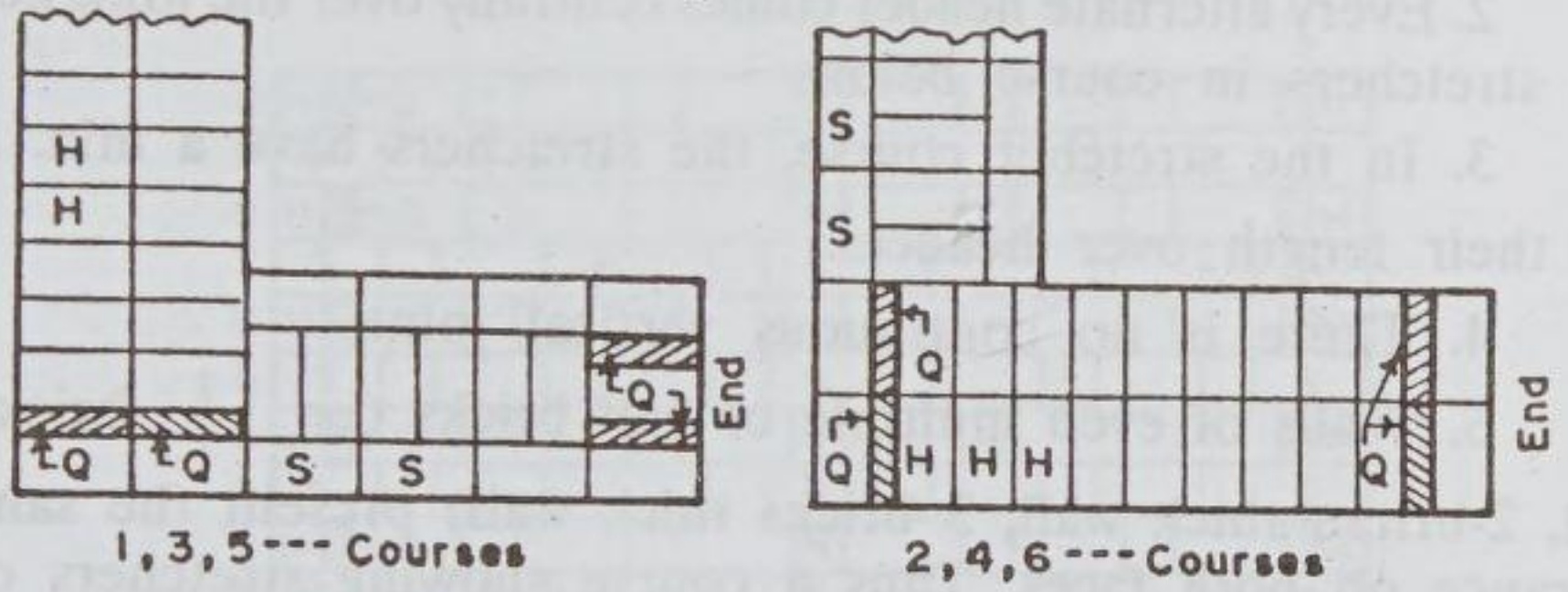
(d) Plan for $2\frac{1}{2}$ Brick Thick Wall

S = STRETCHER FACING ; H = HEADER FACING ; Q = QUEENS CLOSER ; B_1 = QUARTER BAT (QUARTER QUEEN'S CLOSER)

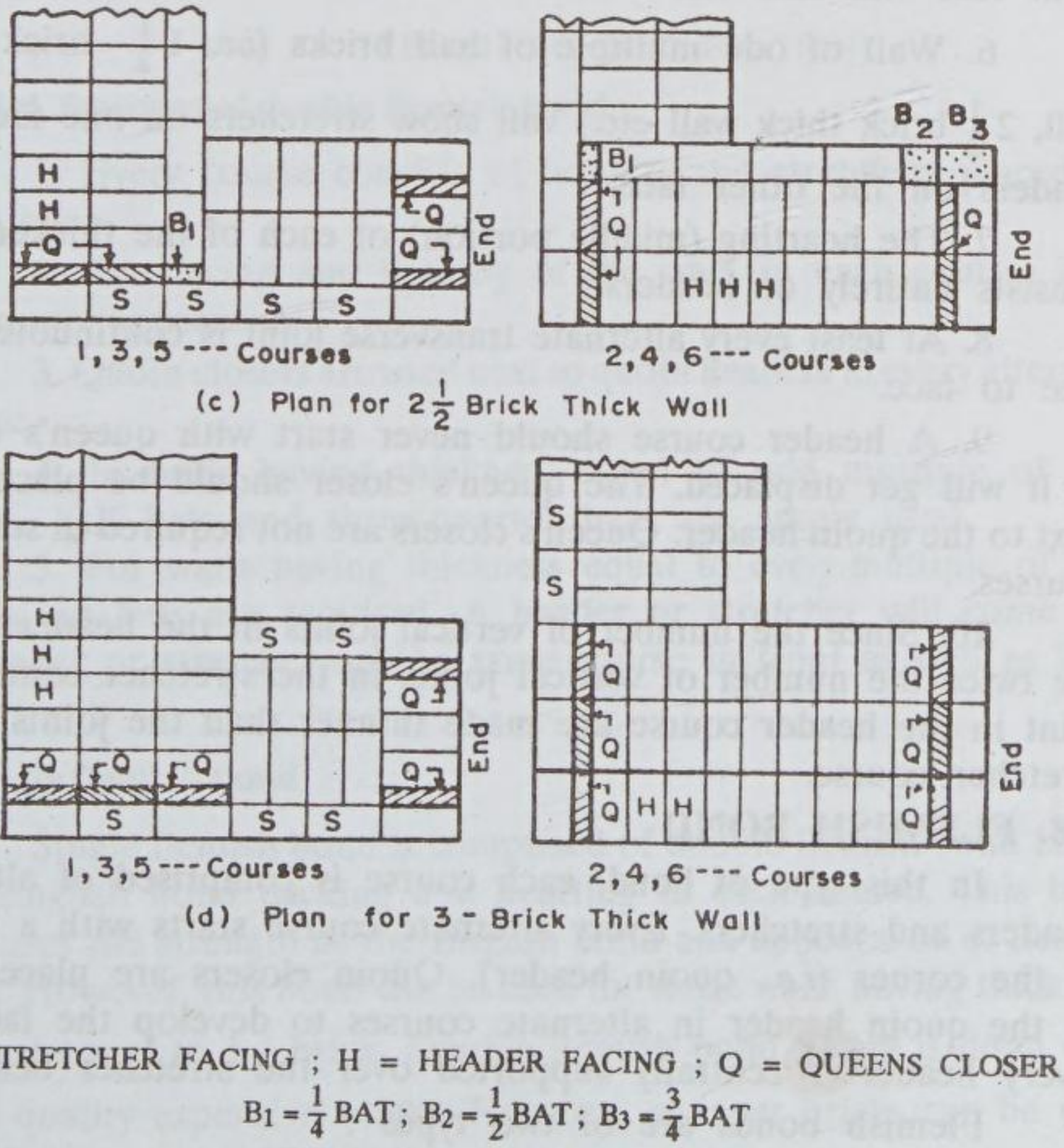
FIG. 6.7. ENGLISH BOND



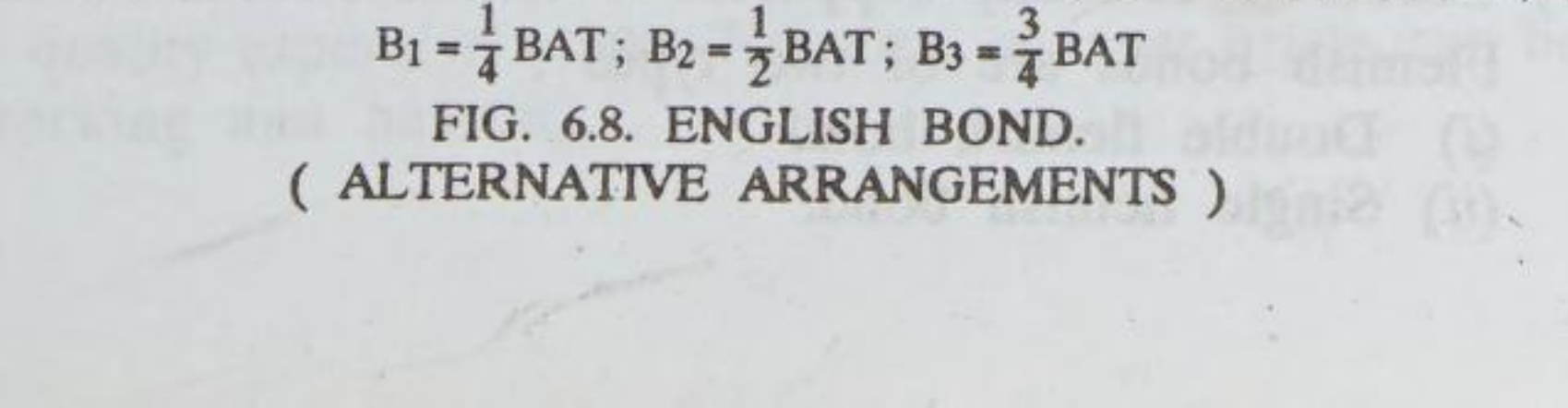
(a) Plan for $1\frac{1}{2}$ Brick Thick Wall



(b) Plan for 2–Brick Thick Wall



(c) Plan for $2\frac{1}{2}$ Brick Thick Wall



(d) Plan for 3 - Brick Thick Wall

S = STRETCHER FACING ; H = HEADER FACING ; Q = QUEENS CLOSER ;

$B_1 = \frac{1}{4}$ BAT ; $B_2 = \frac{1}{2}$ BAT ; $B_3 = \frac{3}{4}$ BAT

FIG. 6.8. ENGLISH BOND.
(ALTERNATIVE ARRANGEMENTS)

in each heading course. Also, only headers are used for the hearting of thicker walls. Fig. 6.6 shows the general elevation of the English bond. Fig. 6.7 and 6.8 shows English bonds for walls of various thicknesses.

Essential Features. Following are the essential features of English bond.

1. Alternative courses will show either headers or stretchers in elevation.

2. Every alternate header comes centrally over the joint between two stretchers in course below.

3. In the stretcher course, the stretchers have a min. lap of $\frac{1}{4}$th their length over headers.

4. There is no continuous vertical joint.

5. Walls of even multiple of half bricks (*i.e.* $1\frac{1}{2}$ brick thick wall, 2-bricks thick wall, 3–bricks thick wall) present the same appearance on both faces. Thus a course showing stretchers on the front face will also show stretchers on the back face.

6. Wall of odd multiple of half bricks (*i.e.* $1\frac{1}{2}$ brick thick wall, $2\frac{1}{2}$ brick thick wall etc.) will show stretchers on one face and headers on the other face.

7. The hearting (middle portion) of each of the thicker walls consists entirely of headers.

8. At least every alternate transverse joint is continuous from face to face.

9. A header course should never start with queen's closer, as it will get displaced. The queen's closer should be placed just next to the quoin header. Queen's closers are not required in stretcher courses.

10. Since the number of vertical joints in the header course are twice the number of vertical joints in the stretcher course, the joint in the header course are made thinner than the joints in the stretcher course.

6.8. FLEMISH BOND

In this type of bond, each course is comprised of alternate headers and stretchers. Every alternate course starts with a header at the corner (*i.e.*. quoin header). Quoin closers are placed next to the quoin header in alternate courses to develop the face up. Every header is centrally supported over the stretcher below it.

Flemish bonds are of two types :

(*i*) Double flemish bond

(*ii*) Single flemish bond.

1. Double flemish bond

In the double flemish bond, each course presents the *same appearance* both in the front face as well as in the back face. Alternate headers and stretcher are laid in each course. Because of this, double flemish bond presents better appearance than English bond. Fig. 6.9 shows the general elevation of flemish bond, for all the wall thicknesses. Fig.6.10 shows the double flemish bond in plan, for walls of various thicknesses.

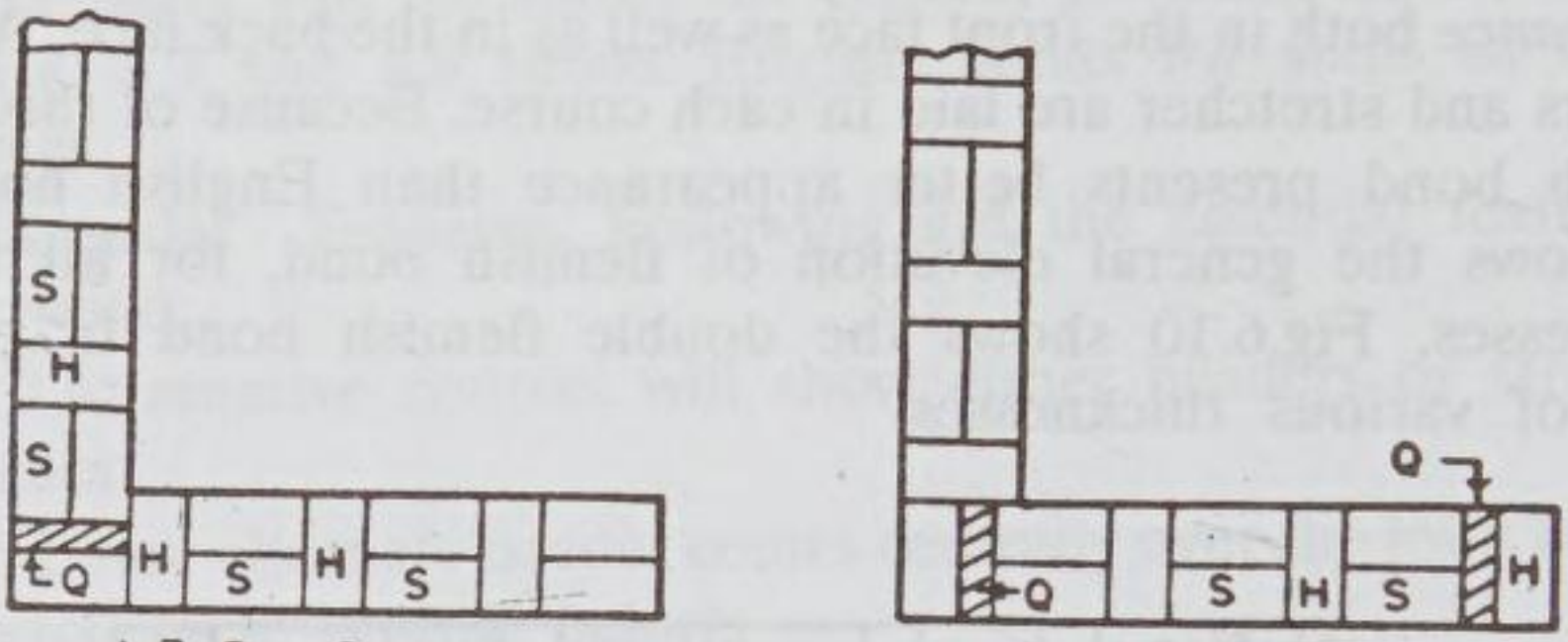
FIG. 6.9. FLEMISH BOND (ELEVATION).

Special features of double flemish bond

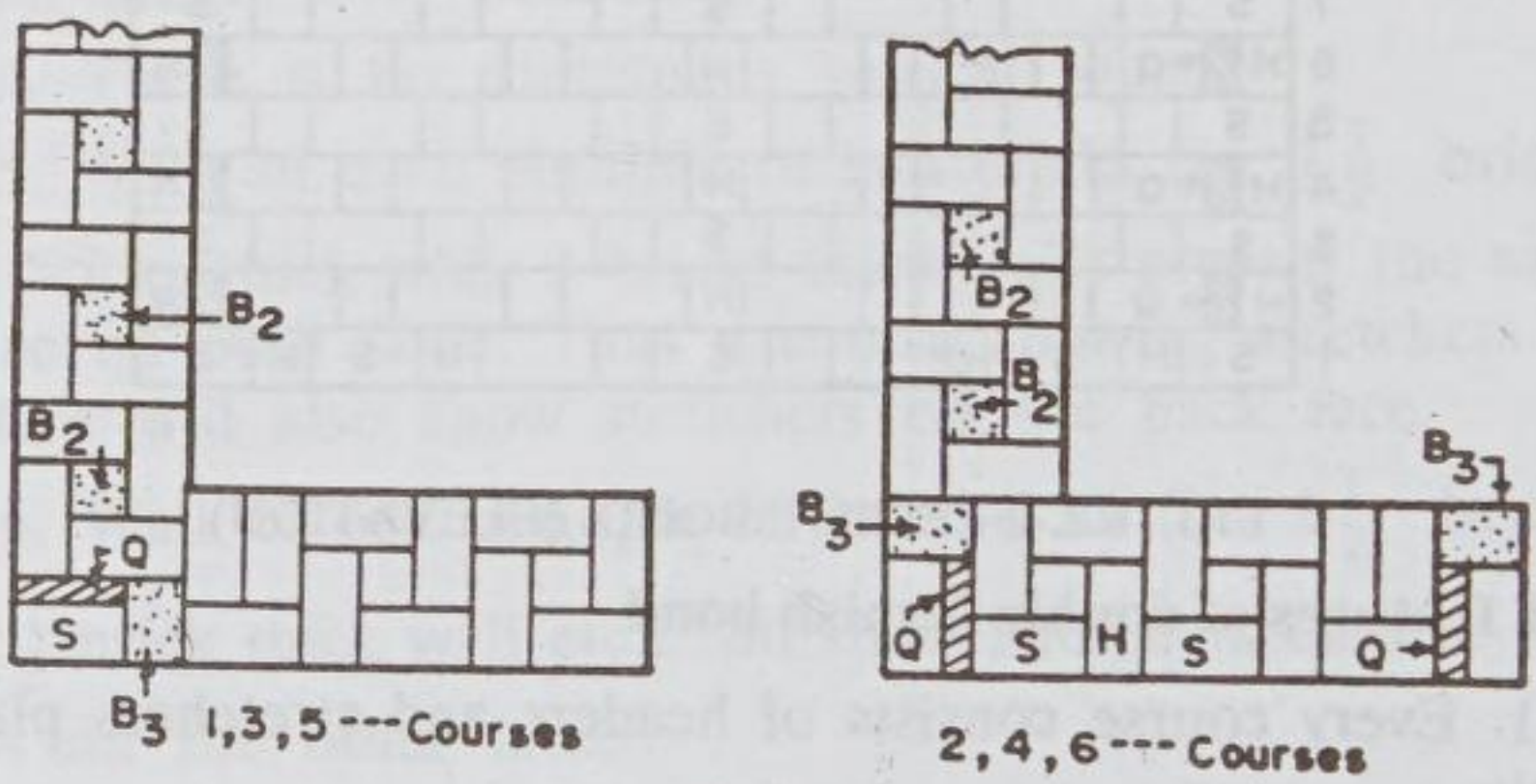
1. Every course consists of headers and stretchers placed alternately.
2. The facing and backing of the wall, in each course, have the same appearance.
3. Quoin closers are used next to quoin headers in every alternate course.
4. In walls having thickness equal to odd multiple of half bricks, half bats and three-quarter bats are amply used.
5. For walls having thickness equal to even multiple of half bricks, no bats are required. A header or stretcher will come out as header or stretcher on the same course in front as well as back faces.

2. Single flemish bond

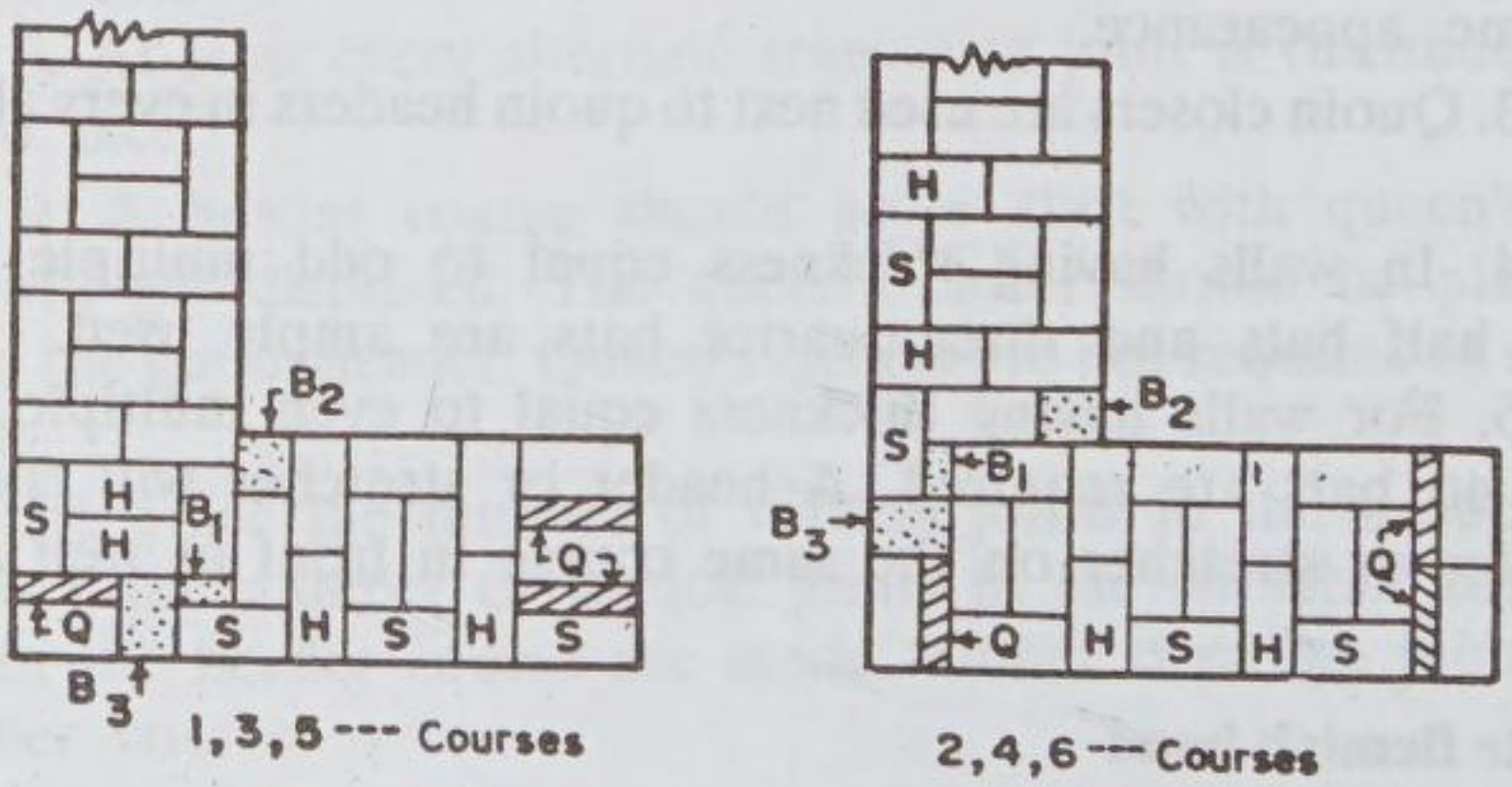
Single flemish bond is comprised of double flemish bond facing and English bond backing and hearting in each course. This bond thus uses the strength of the English bond and appearance of flemish bond. However, this bond can be used for those walls having thickness at least equal to $1\frac{1}{2}$ brick. Double flemish bond facing is done with good quality expensive bricks. However, cheaper bricks can be used for backing and hearting.



(a) Plan for one Brick Thick Wall



(b) Plan for $1\frac{1}{2}$ Brick Thick Wall

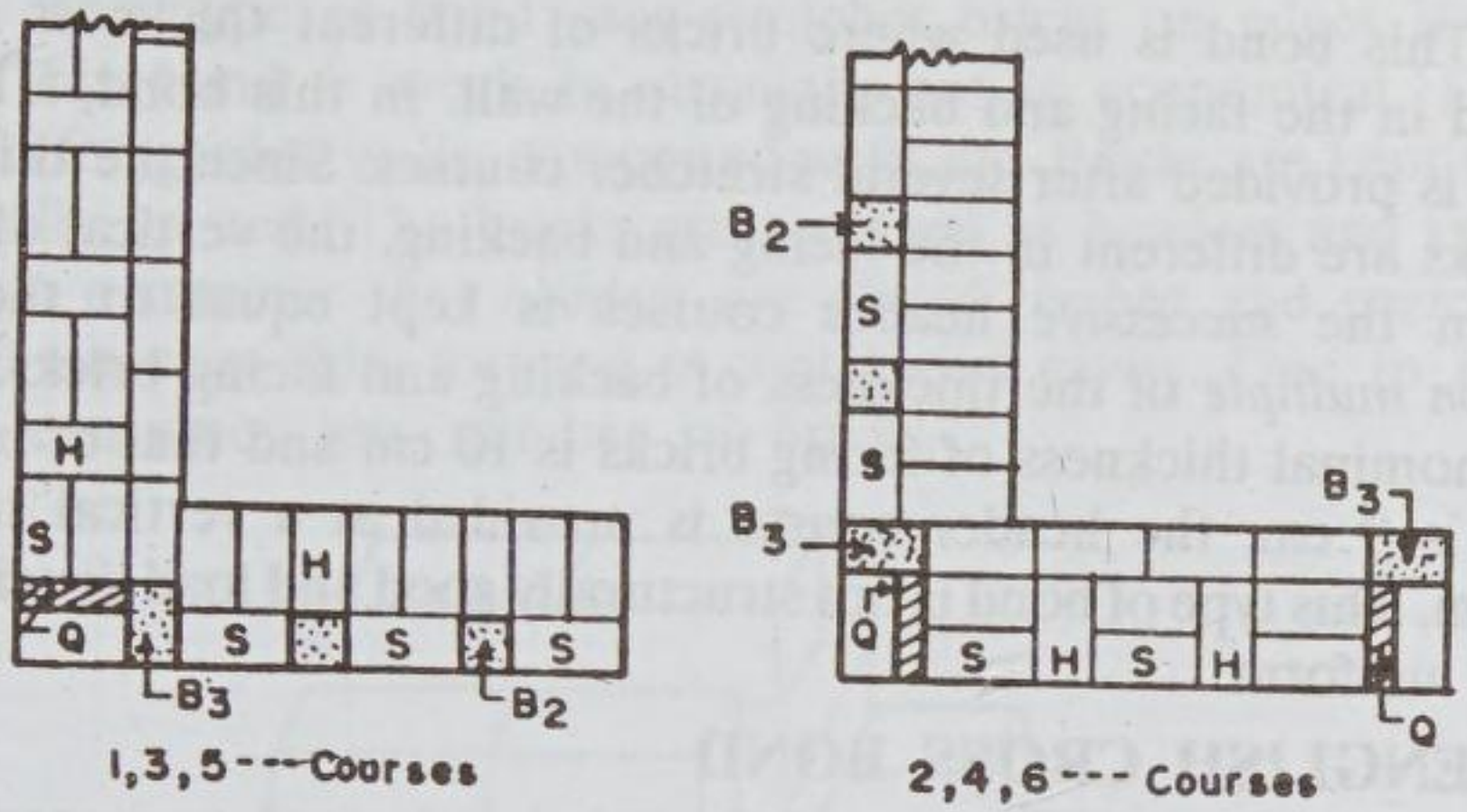


(c) Plan for 2 Brick Thick Wall

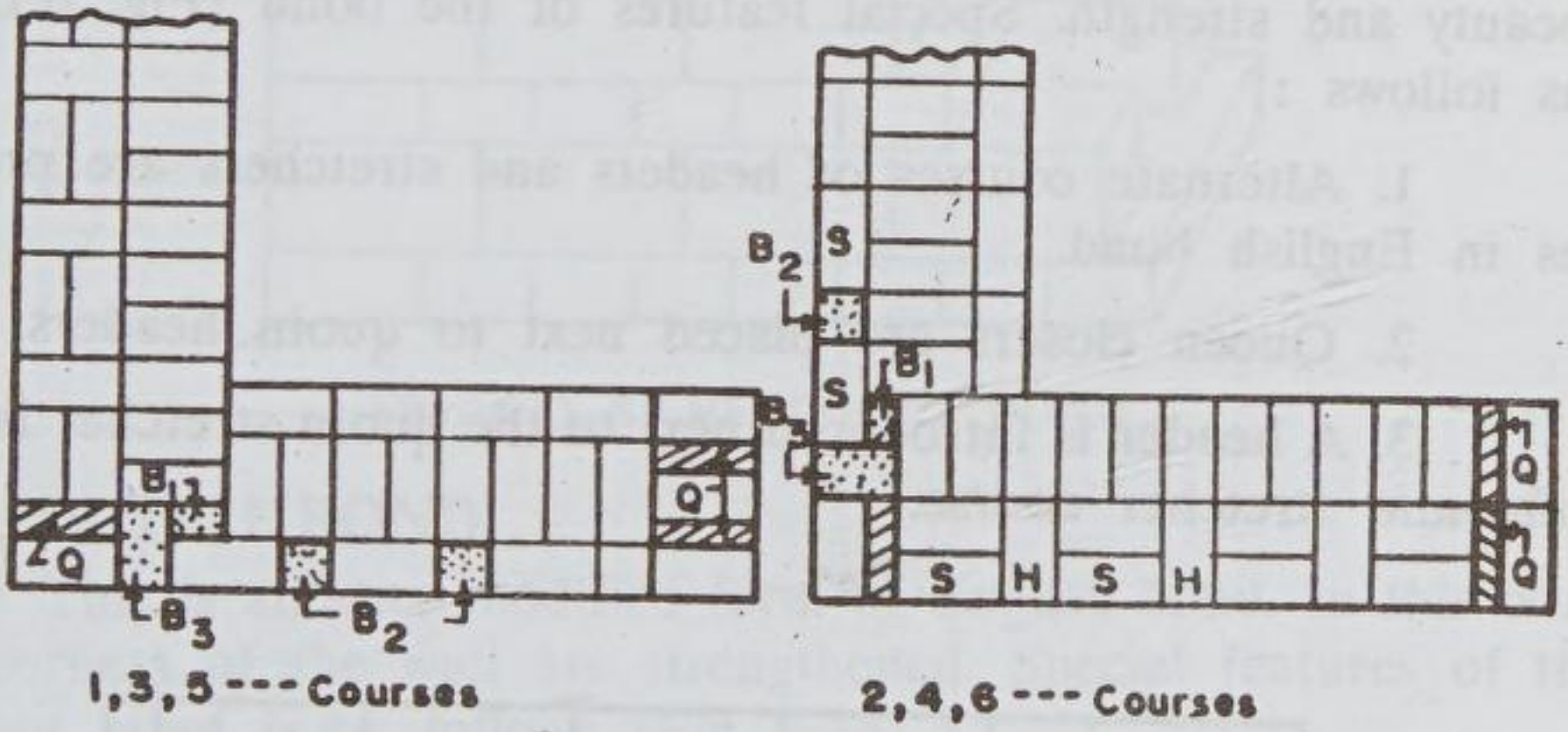
S = STRETCHER ; H = HEADER ; Q = QUEENS CLOSER ;
B_2 = HALF BAT ; $B_3 = \frac{3}{4}$ BRICK ; B_1 = QUARTER BAT

FIG. 6.10. DOUBLE FLEMISH BOND.

Fig. 6.11 shows the plan of single flemish bond for various thicknesses of the wall.



(a) Plan for $1\frac{1}{2}$ Brick Thick Wall



(b) Plan for 2 Brick Thick Wall

S = STRETCHER ; Q = QUEENS CLOSER ;

B_2 = HALF BAT ; $B_3 = \frac{3}{4}$ BRICK ; B_1 = QUARTER BAT

FIG. 6.11. SINGLE FLEMISH BOND.

Comparison of English Bond and Flemish Bond

1. English bond is stronger than flemish bond for walls thicker than $1\frac{1}{2}$ brick.

2. Flemish bond gives more pleasing appearance than the English bond.

3. Broken bricks can be used in the form of bats in Flemish bond. However, more mortar is required.

4. Construction with Flemish bond requires greater skill in comparison to English bond.

6.9. FACING BOND

This bond is used where bricks of different thickness are to be used in the facing and backing of the wall. In this bond, a header course is provided after several stretcher courses. Since the thickness of bricks are different in the facing and backing, the vertical distance between the *successive* header courses is kept equal to the *least common multiple* of the thickness of backing and facing bricks. Thus, if the nominal thickness of facing bricks is 10 cm and that of backing bricks is 9 cm, the header course is provided at a vertical interval of 90 cm. This type of bond is not structurally good and load distribution is not uniform.

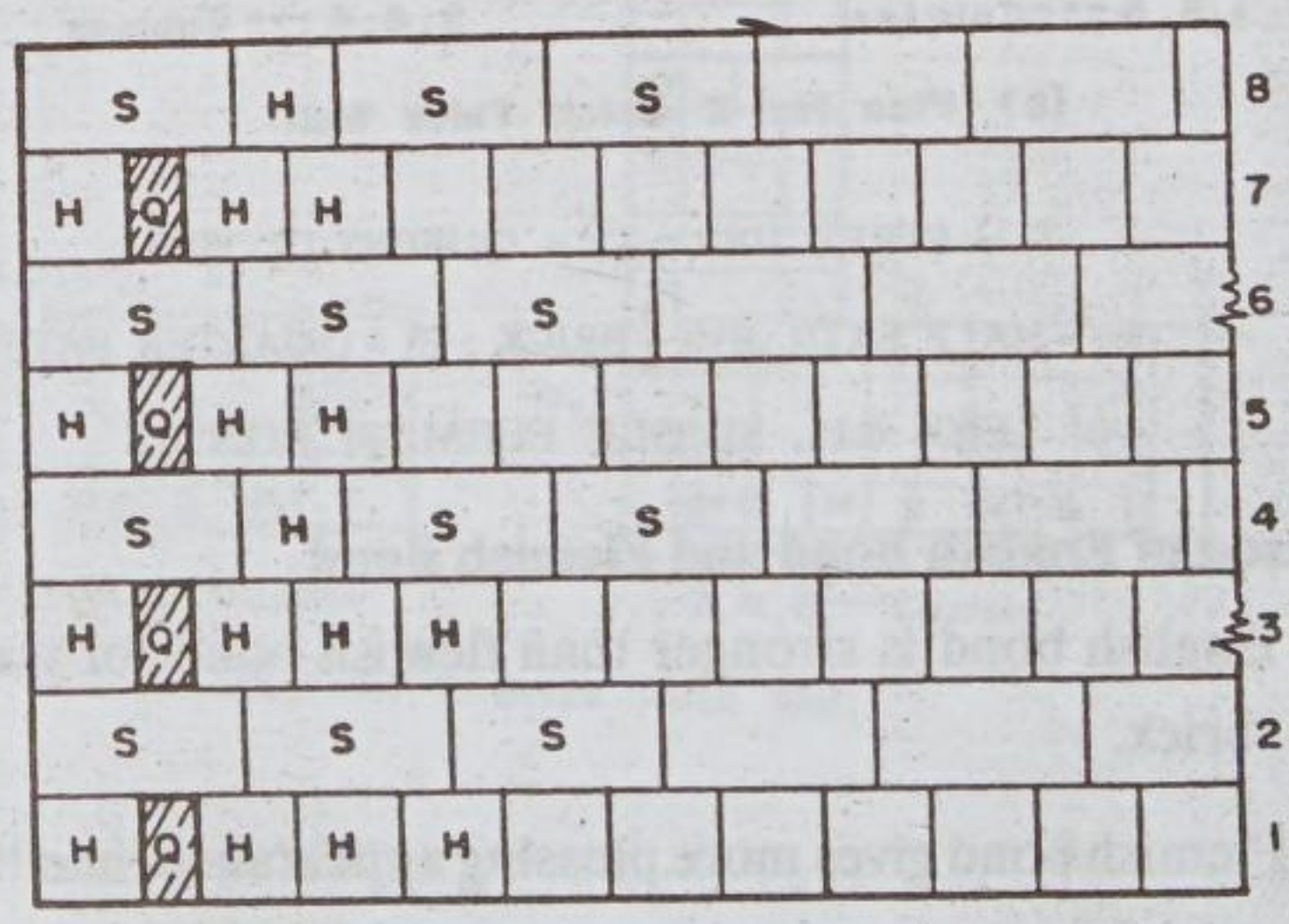
6.10. ENGLISH CROSS BOND

This is a modification of English bond, used to improve the appearance of the wall. This bond combines the requirements of beauty and strength. Special features of the bond (Fig. 6.12) are as follows :

1. Alternate courses of headers and stretchers are provided as in English bond.

2. Queen closers are placed next to quoin headers.

3. A header is introduced next to the quoin stretcher in every *alternate* stretcher course.



H = HEADER ; S = STRETCHER ; Q = QUEEN'S CLOSER

FIG. 6.12. ENGLISH CROSS BOND

6.11. BRICK ON EDGE BOND (SILVERLOCK'S BOND OR SOLDIER'S COURSE)

This type of bond uses stretcher bricks on edges instead of bed. This bond is weak in strength, but is economical. Hence it is used for garden walls, compound walls etc. Bricks are kept standing vertically on end. The bricks are arranged as headers and stretchers in such a manner that *headers are placed on bed* and *stretchers are placed on edge* thus forming a continuous cavity. Due to this, the bond consumes less number of bricks.

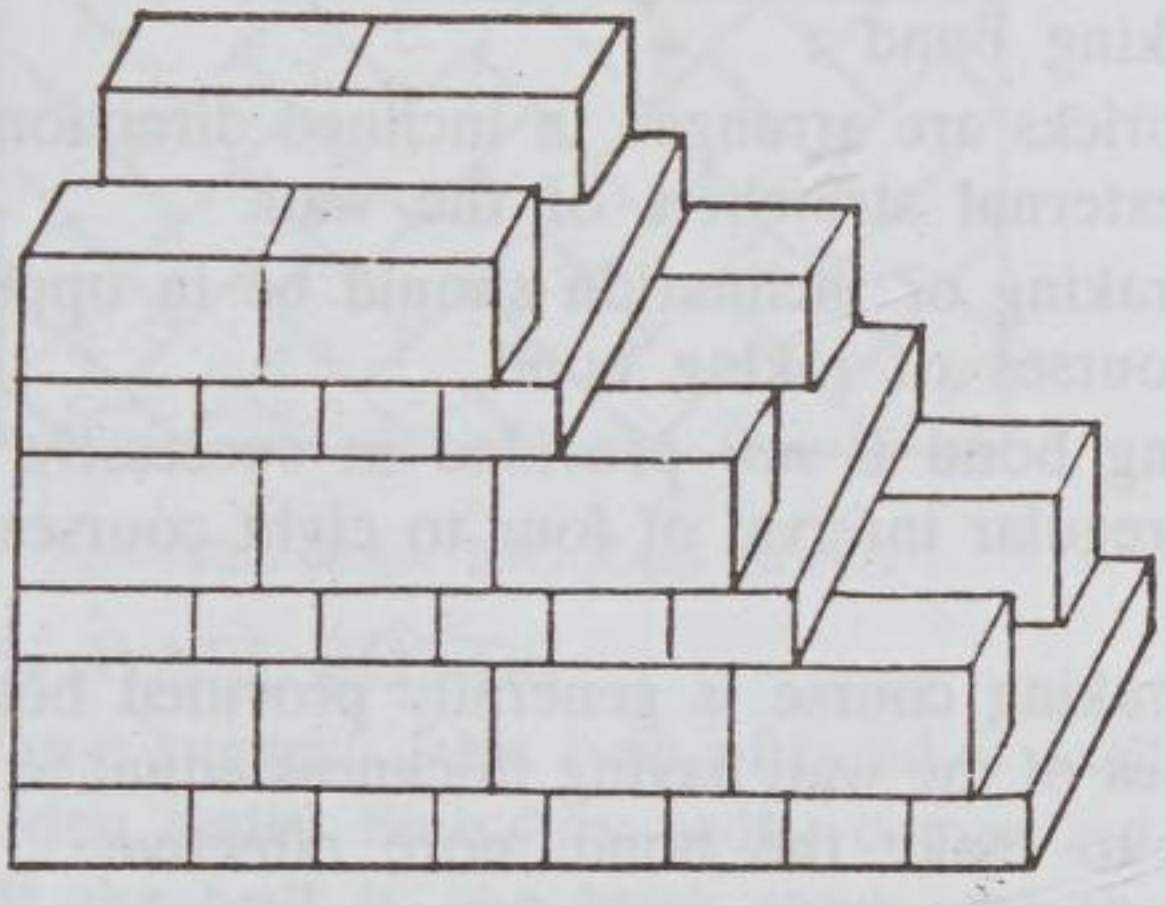
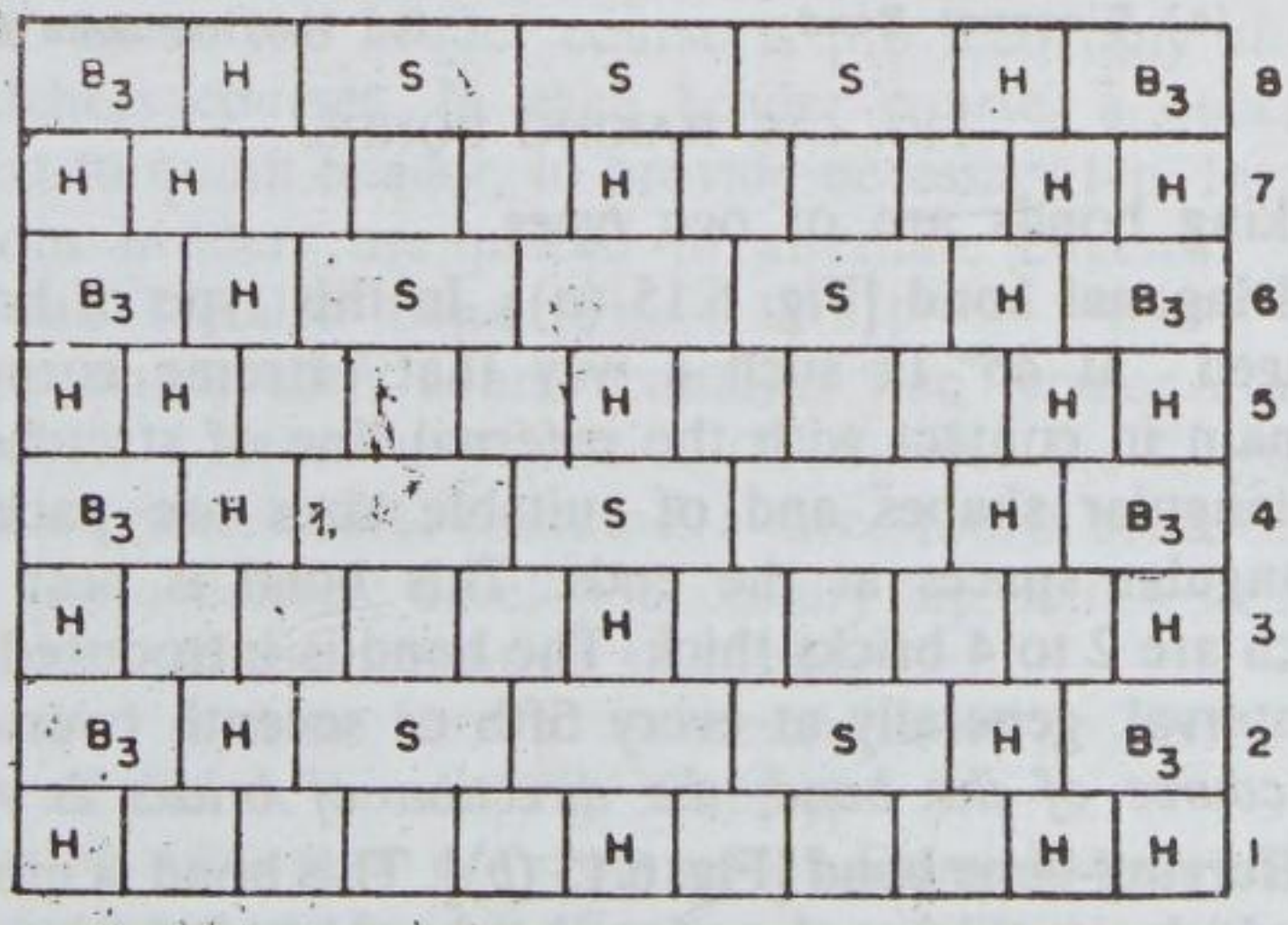


FIG. 6.13. SILVERLOCK'S BOND.

6.12. DUTCH BOND

This is another modified form of English bond. In this bond the corners of the wall are strengthened. Special features of this type of bond is as follows (Fig. 6.14) :

1. Alternate courses of headers and stretchers are provided as in English bond.



H = HEADER ; S = STRETCHER ; B_3 = 3/4 BRICK BAT.

FIG. 6.14. DUTCH BOND.

2. Every stretcher course starts at the quoin with a three-quarter bat.

3. In every *alternate* stretcher course, a header is placed next to the three-quarter brick bat provided at the quoin.

6.13. RAKING BOND

This bond is used in thick walls. In this type of bond, the bonding bricks are kept at an inclination to the direction of the wall. Due to this, the longitudinal stability of thick wall built in English bond is very much increased. This bond is introduced at certain intervals along the height of the wall. Following are special features of raking bond :

1. The bricks are arranged in inclined direction, in the space between the external stretchers of the wall.

2. The raking or inclination should be in opposite direction in alternate courses of raking bond.

3. Raking bond is *not* provided in successive courses. It is provided at a regular interval of four to eight courses in the height of a wall.

4. The raking course is generally provided between the two stretcher courses of the wall having thickness equal to even multiple of half-bricks, to make the bond more effective.

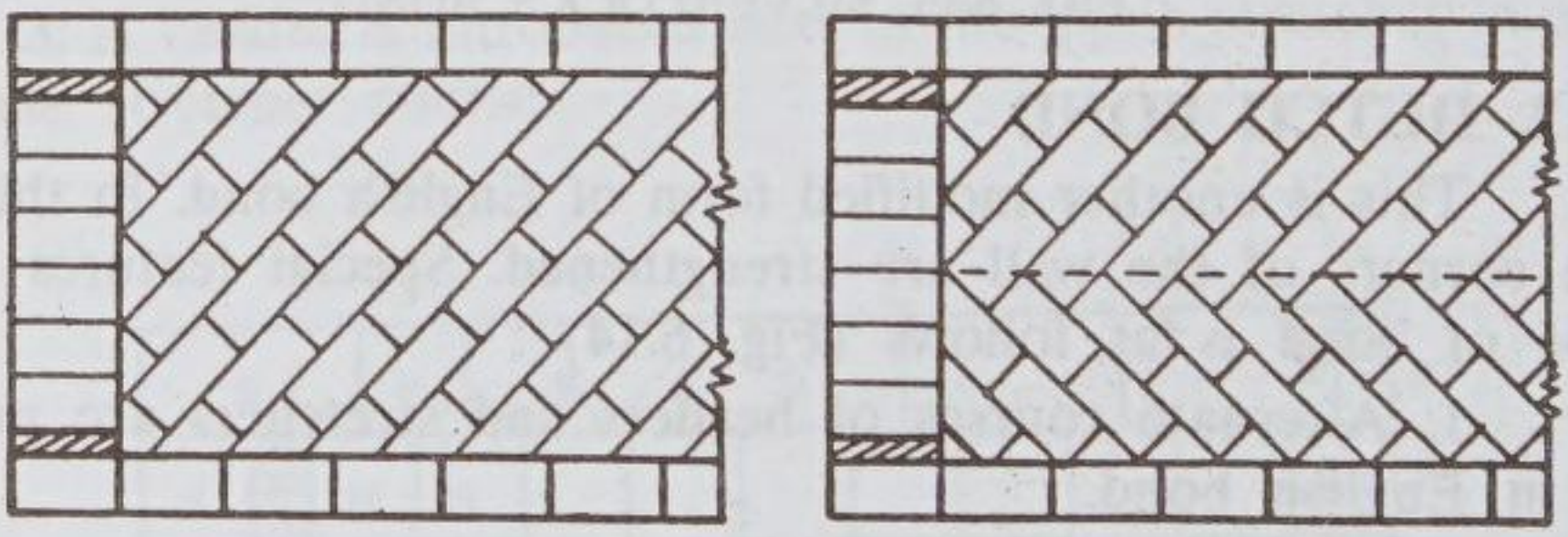


FIG. 6.15. RAKING BONDS.

Raking bonds are of *two types* :

1. Diagonal bond [Fig. 6.15 (*a*)]. In this type of bond, bricks are arranged at 45° in such a way that extreme corners of the series remain in contact with the external line of stretchers. Bricks cut to triangular shapes and of suitable sizes are packed in the small triangular spaces at the ends. This bond is best suited for walls which are 2 to 4 bricks thick. The bond is introduced at regular vertical interval, generally at every fifth or seventh course. *In every alternate course of the bond, the direction of bricks is reversed.*

2. Herring-bone bond [Fig. 6.15 (*b*)]. This bond is more suitable for walls which are thicker than four bricks thick. Bricks are arranged

at 45° in two opposite directions from the centre of the wall thickness, as shown in [Fig. 6.15 (*b*)]. The bond is introduced in the wall at regular vertical interval. *In every alternate course, the directions of bricks are changed.* The bond is also used for ornamental finish to the face work, and also for brick flooring.

6.14. ZIG ZAG BOND

This bond is similar to herring-bone bond, except that the bricks are laid in zig-zag fashion, as shown in Fig. 6.16. This bond is commonly used for making ornamental panels in the brick flooring.

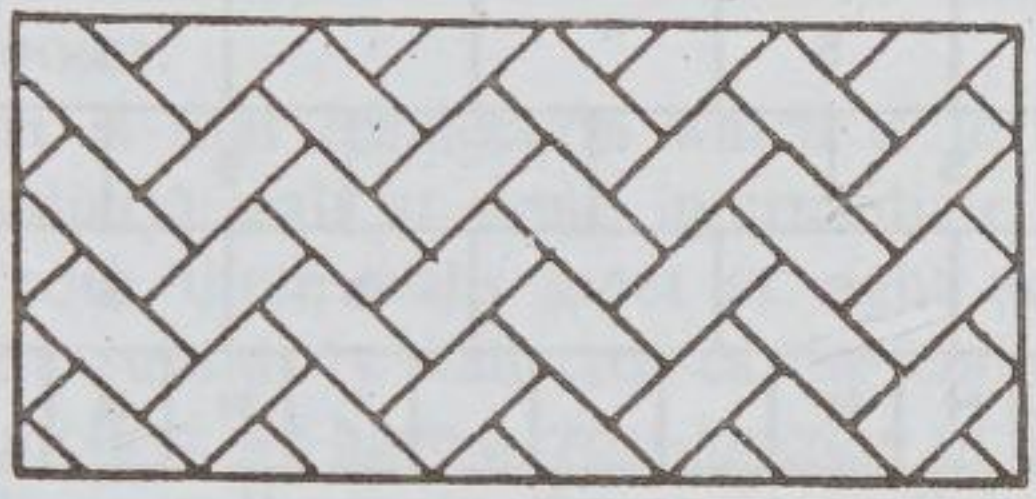


FIG. 6.16. ZIG-ZAG BOND.

6.15. GARDEN WALL BONDS

As the name suggests, this type of bond is used for the construction of garden walls, boundary walls, compound walls, where the thickness of the wall is one brick thick and the height does not exceed two metres. This type of bond is not so strong as English bond, but is more attractive. Due to this reason, it is sometimes used in the construction of outer leaves of cavity walls.

Garden wall bonds are of three types :

(*i*) Garden wall English bond
(*ii*) Garden wall Flemish bond
(*iii*) Garden wall Monk bond.

1. Garden wall English bond [Fig. 6.17(*b*)]

In this bond, the header course is provided only after three to five stretchers courses. In each header course, a queen closer is placed next to quoin header, to provide necessary lap. In stretcher courses, quoin headers are placed in alternate courses.

2. Garden wall Flemish bond [Fig. 6.17(*c*)]

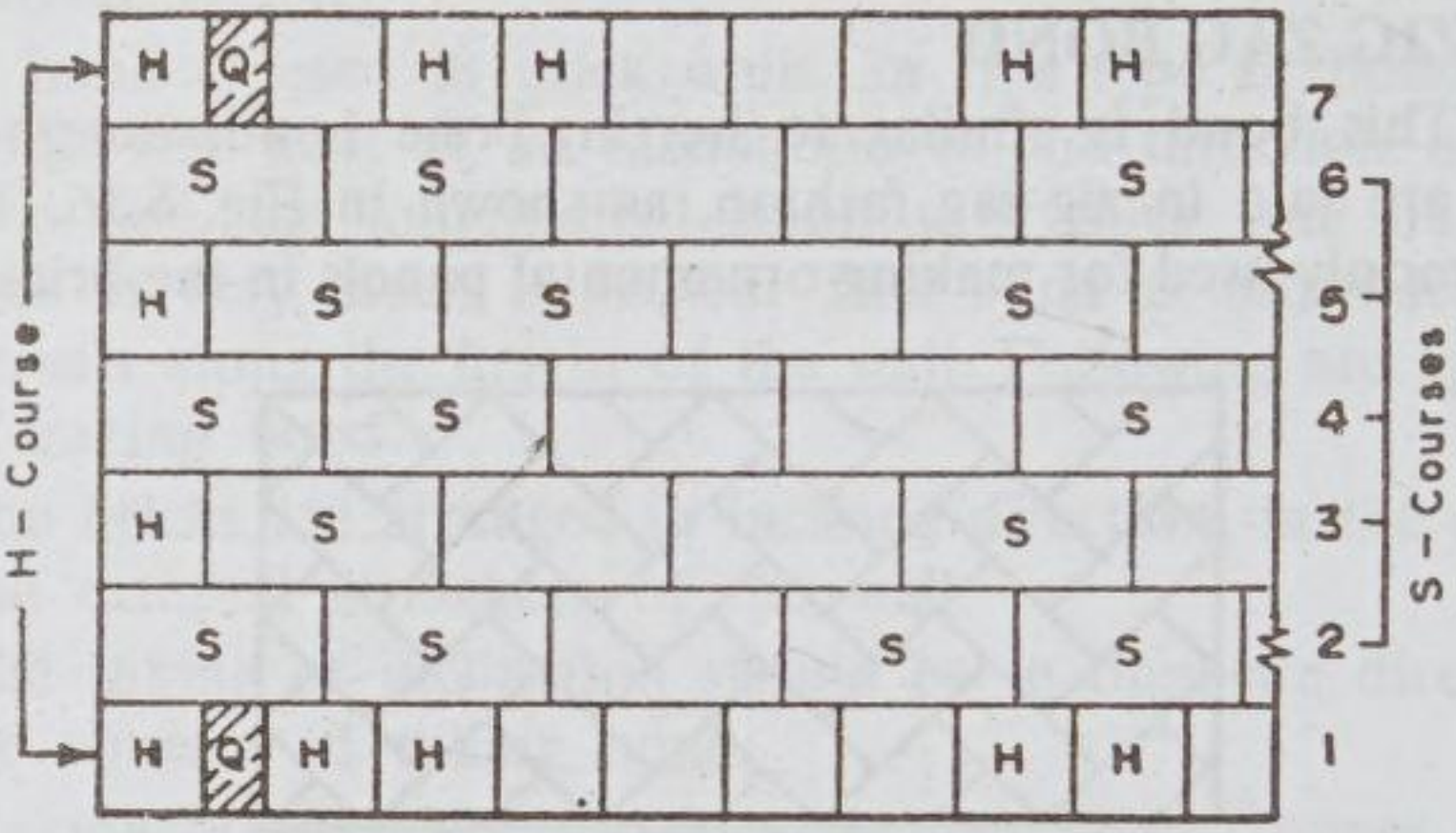
In this bond, each course contains one header after three to five stretchers continuously placed, throughout the length of the course. Each alternate course containes a three-fourth brick bat placed next to the quoin header, develop necessary lap, and a header laid over the middle of each central stretcher.

This bond is also known as *scotch bond or sussex bond.*

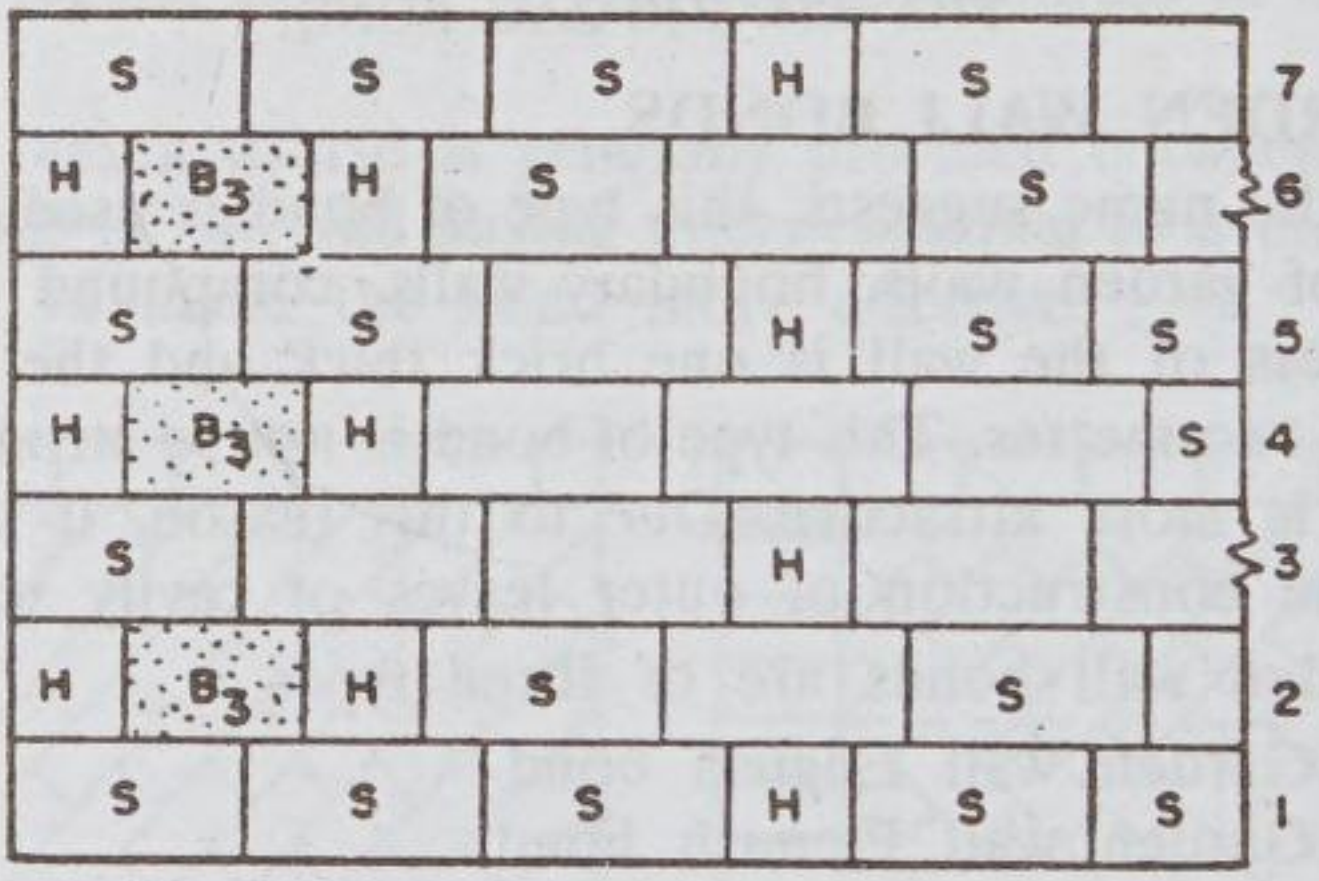
3. Garden wall Monk bond [Fig. 6.17(*c*)]

This is special type of garden-wall Flemish bond in which each course contains one header after two successive stretchers. Every

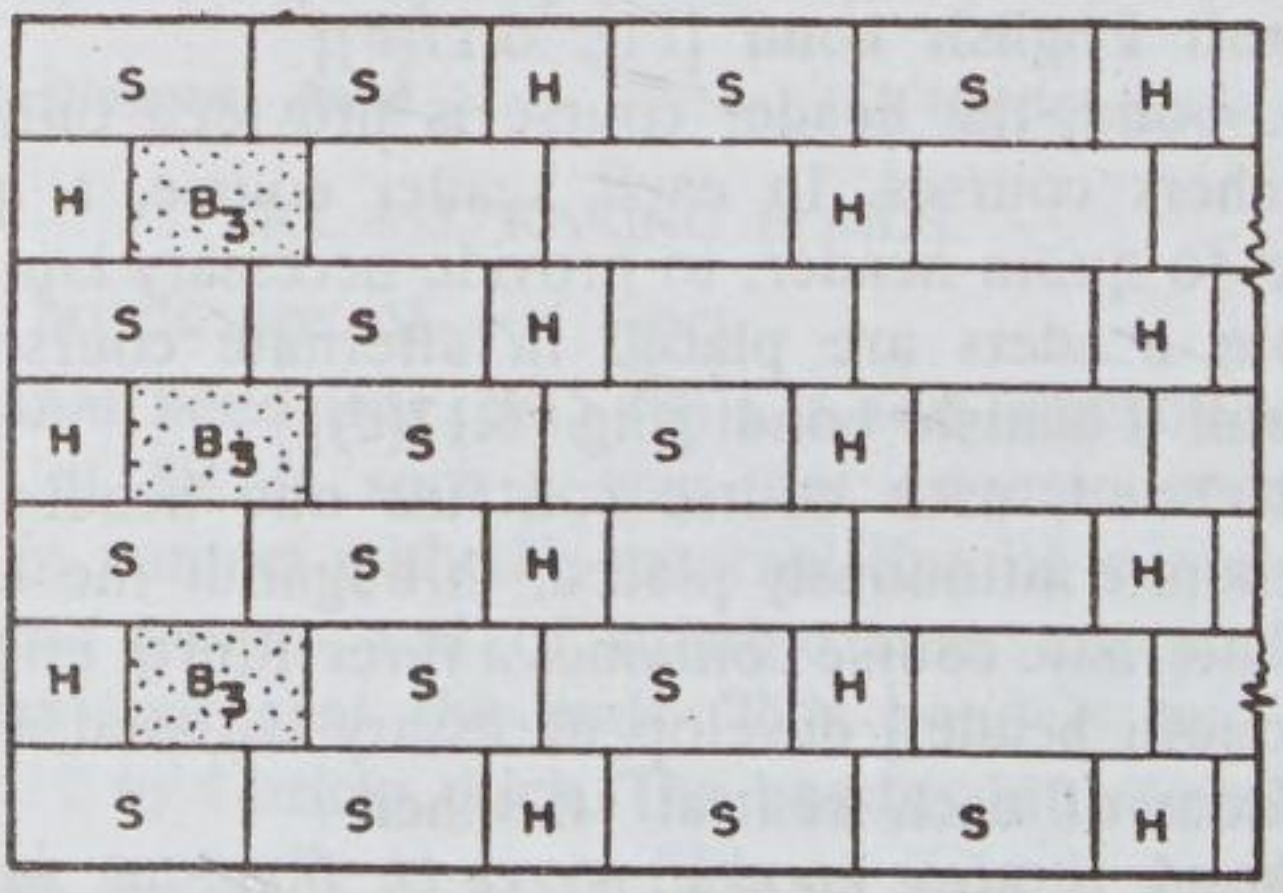
alternate course contains a quoin header followed by a $\frac{3}{4}$ brick bat. Due to this, the header rests over the joint between two successive stretchers.



(a) Garden Wall English Bond



(b) Garden Wall Flemish Bond



(c) Monk Bond

S = STRETCHER ; H = HEADER ; $B_3 = \frac{3}{4}$ BRICK BATS

FIG. 6.17. GARDEN WALL BONDS.

6.16. BOND AT CONNECTIONS

Connection is the place where two walls coming from different directions meet. The walls should be properly *united* at the connecting point through some proper *bond.* The following three requirements should be satisfied by the bond at the connection. (*i*) There should be no continuity in the vertical joints, (*ii*) use of brick bats should be as minimum as possible, and (*iii*) the connection should be structurally strong to resist differential settlement, if any.

Connections are of the following two types :

(*a*) Junctions

(*b*) Quoins.

Junction is that connection which is formed at the meeting of one (subsidiary) wall at same intermediate position of another wall. When both these walls meet at right angles, we get a *tee-junction*. If the subsidiary wall crosses the main wall and continues beyond the junction, we have a *cross-junction* or *intersection.* However, if the subsidiary wall meets the main wall at some intermediate point, and if the angle formed between the two is other than a right angle, a *squint junction* is formed.

Quoin is the connection formed when two external walls meet. Alternatively, quoin is the connection which is formed when a wall takes a turn. When the two walls meet at 90°, we have a *right angled* or *square quoin*. If the angle at the connection is other than 90°, a *squint quoin* is formed.

(A) JUNCTIONS

Junctions are of the following types :

(*a*) ***Right-angled junction***

(*i*) Tee-junction

(*ii*) Intersection or cross-junction.

(*b*) *Squint-junction.*

1. (*i*) Tee-junction

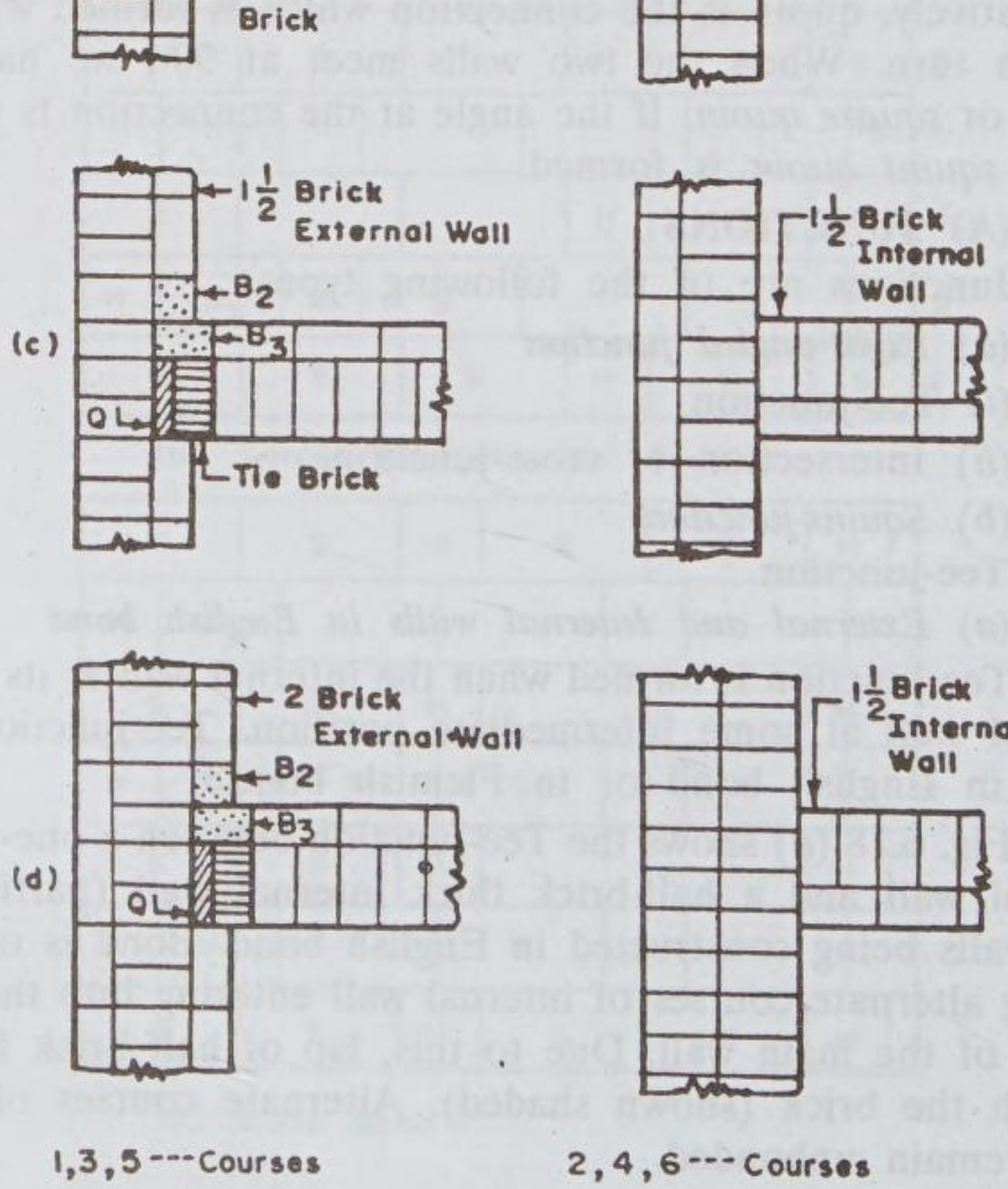
(*a*) *External and Internal walls in English bond*

Tee-junction is formed when the internal wall at its end meets external wall at some intermediate position. Tee-junctions can be either in English bond or in Flemish bond.

Fig. 6.18 (*a*) shows the Tee-junction between a one-brick thick external wall and a half-brick thick internal wall (partition wall), both walls being constructed in English bond. Bond is obtained by making alternate courses of internal wall entering into the stretcher course of the main wall. Due to this, lap of half brick is obtained through the brick (shown shaded). Alternate courses of both the walls remain unbonded.

Fig. 6.18 (*b*) shows the Tee-junction between $1\frac{1}{2}$ brick thick

external wall and one-brick thick internal (cross) wall, both the walls being constructed in English bond. Here, the header course of the internal wall centres the stretcher course of the main wall through half of its width. Due to this, lap of quarter-brick is obtained through the tie-brick, which is placed near the queen closer (Q). Alternate courses of both the walls remain unbonded.



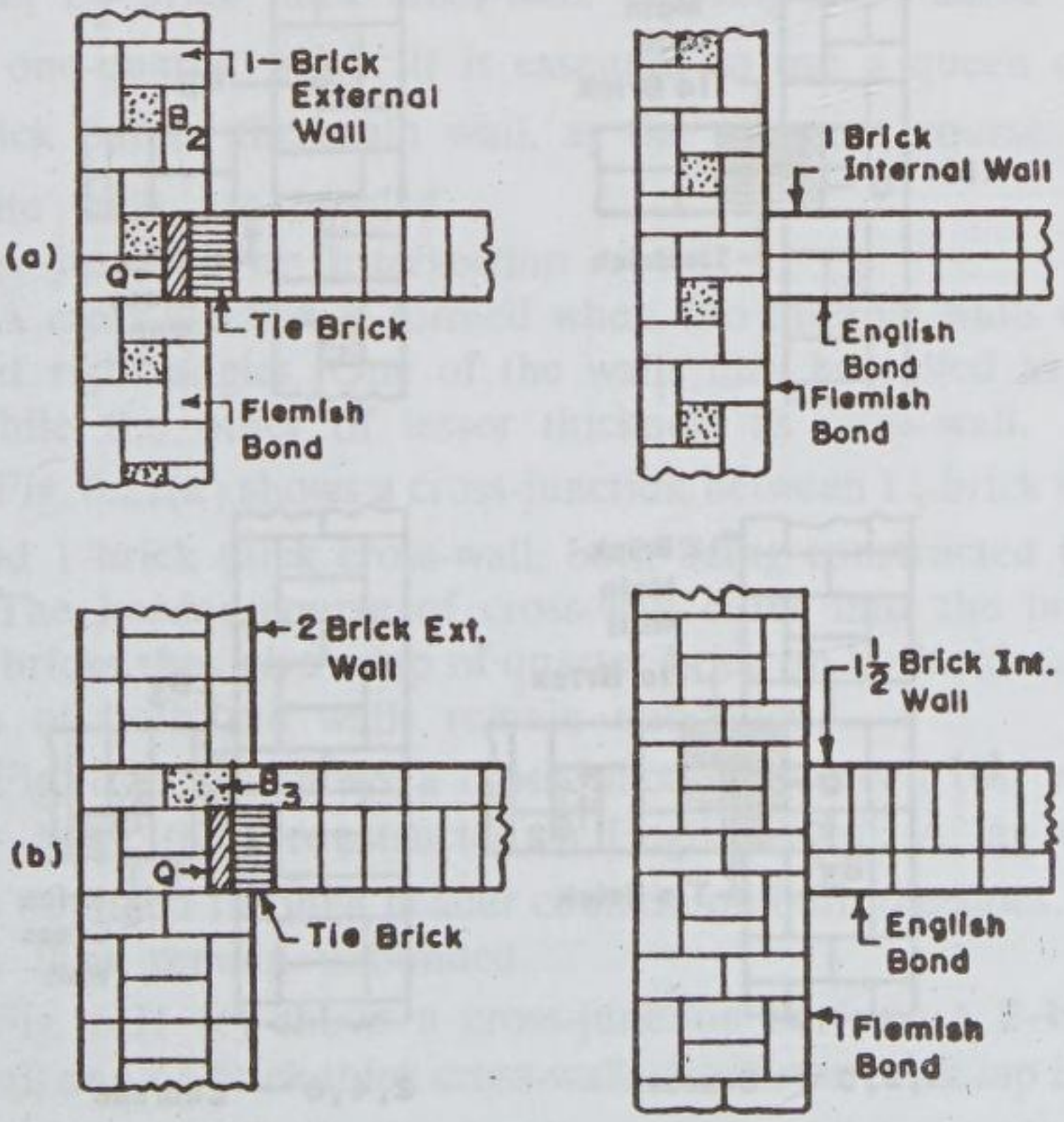
B_2 = HALF BRICK ; $B_3 = \frac{3}{4}$ BRICK

FIG. 6.18. T-JUNCTIONS IN ENGLISH BOND

Fig. 6.18 (*c*) shows the Tee-junction between $1\frac{1}{2}$ brick thick external wall and $1\frac{1}{2}$ brick thick internal wall, both the walls being constructed in English bond. In alternate courses, the header brick at the junction enters the stretcher course of the main wall. The tie-brick (shown shaded), placed near the queen closer (Q) furnishes a lap of quarter brick. Additional lap is obtained in the same course, through placing a $\frac{3}{4}$ brick bat as shown. Alternate courses of both the walls remain unbonded.

Fig 6.18 (*d*) shows the Tee-junction between 2-brick thick main wall and $1\frac{1}{2}$ brick thick cross-wall, both the walls being constructed in English bond. Here, the header course of internal wall enters the stretcher course of the main wall through half of its width. Due to this, lap of quarter brick is obtained through the tie-brick (header brick) which is placed near the queen closer (Q). Additional lap is obtained in the same course, through placing a $\frac{3}{4}$ brick bat as shown. Here also, alternate courses of both the walls remain unbonded.

(b) External wall in Flemish bond and Internal wall in English bond.

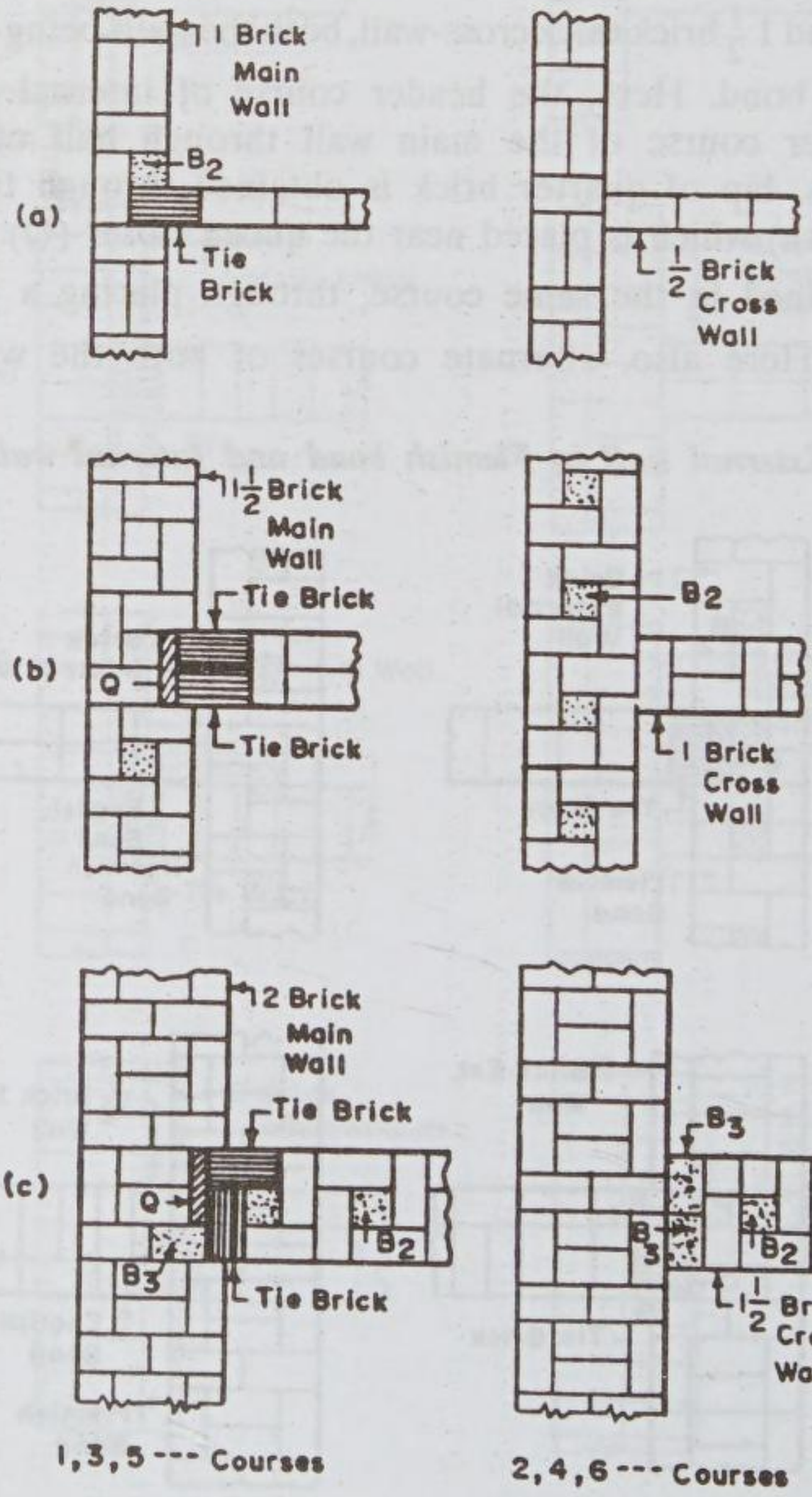


$B_2 = \frac{1}{2}$ BRICK ; $B_3 = \frac{3}{4}$ BRICK

FIG. 6.19. TEE-JUNCTION FOR EXTERNAL WALL IN FLEMISH BOND AND INTERNAL WALL IN ENGLISH BOND.

Fig. 6.19 (*a*) shows the Tee-junction for a brick thick external wall in Double Flemish bond and one brick thick internal wall in English bond. The header course of internal wall enters into the main wall, thus getting a lap of one-quarter brick. The tie–brick (header course) is placed adjacent to a queen closer. Alternate courses of both the walls remain unbonded.

Fig. 6.19(*b*) shows the Tee-junction for 2–brick thick external wall in Double–Flemish bond and $1\frac{1}{2}$ brick thick internal wall in English bond. Here also, the header course of the cross-wall (internal



B_2= HALF BRICK ; B_3 = 3/4 BRICK ; Q=QUEEN CLOSER.

FIG. 6.20 TEE-JUNCTION IN DOUBLE FLEMISH BOND.

wall) enters the main wall, thus getting a lap of quarter brick. The tie-brick (header brick) is placed next to a queen closer. Additional lap is obtained through the stretcher brick of the same course of the internal wall, which is placed adjacent to a $\frac{3}{4}$ brick bat of the main wall. The alternate courses of both the walls remain unbonded.

(*c*) ***Both external and internal walls in double Flemish bond***

Fig. 6.20 (*a*) shows the Tee-junction for a 1–brick thick main wall and $\frac{1}{2}$ brick thick cross wall, both being constructed in Double Flemish bond. The stretcher bricks of alternate courses of the cross-wall enter into the main wall through half brick length. Due to this, it is necessary to place a half-brick bat adjacent to it, in the main wall. The alternate courses of each wall remain unbonded.

Fig. 6.20 (*b*) shows the tee-junction for a $1\frac{1}{2}$ brick thick main wall and one-brick thick cross-wall, both being constructed in double Flemish bond. In alternate courses, the stretcher bricks of the cross wall enter into the main wall through quarter brick. A queen closer (Q) is placed next to it in the main wall as shown. Alternate courses of both the walls remain unbonded.

Fig. 6.20 (*c*) shows the tee-junction for two-brick thick main wall and $1\frac{1}{2}$ brick thick cross-wall. Bonding is obtained through a lap of one-quarter brick. It is essential to use a queen closer and a $\frac{3}{4}$ brick bat in the main wall, at the alternate courses in which both the walls are bonded.

2. Cross-junction or Intersection

A *cross-junction* is formed when two internal walls cross each other at right angles. One of the walls may be called as the main wall while the other of lesser thickness as cross-wall.

Fig. 6.21(*a*) shows a cross-junction between $1\frac{1}{2}$-brick thick main wall and 1 brick thick cross-wall, both being constructed in English bond. The header course of cross-wall enter into the main wall : the tie bricks thus give a lap of quarter brick on both sides. Alternate courses of both the walls remain unbonded.

Fig. 6.21 (*b*) shows a cross-junction between two walls, each of $1\frac{1}{2}$ -brick thick constructed in English bond. A lap of quarter brick is obtained through header courses, on both the sides. Alternate courses thus remain unbonded.

Fig. 6.21 (*c*) shows a cross-junction between a 2–brick thick main wall and $1\frac{1}{2}$ brick thick cross-wall. A quarter-brick lap is obtained on both sides through the header course. Additional lap is also obtained through $\frac{1}{4}$ stretcher brick on one side and $\frac{3}{4}$–stretcher brick

on the other side. Alternate courses of both the walls remain unbonded.

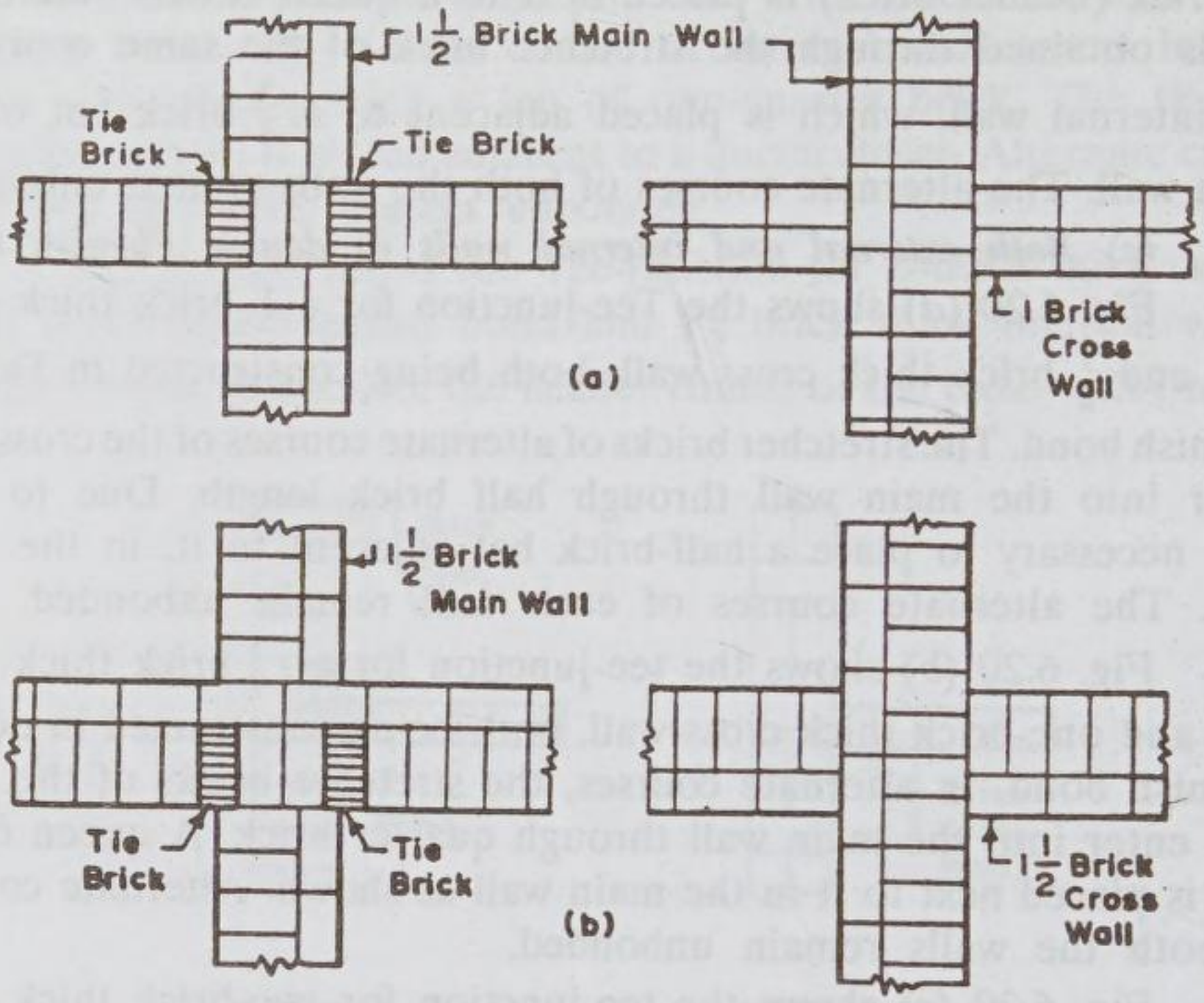


FIG. 6.21. CROSS-JUNCTION IN ENGLISH BOND.

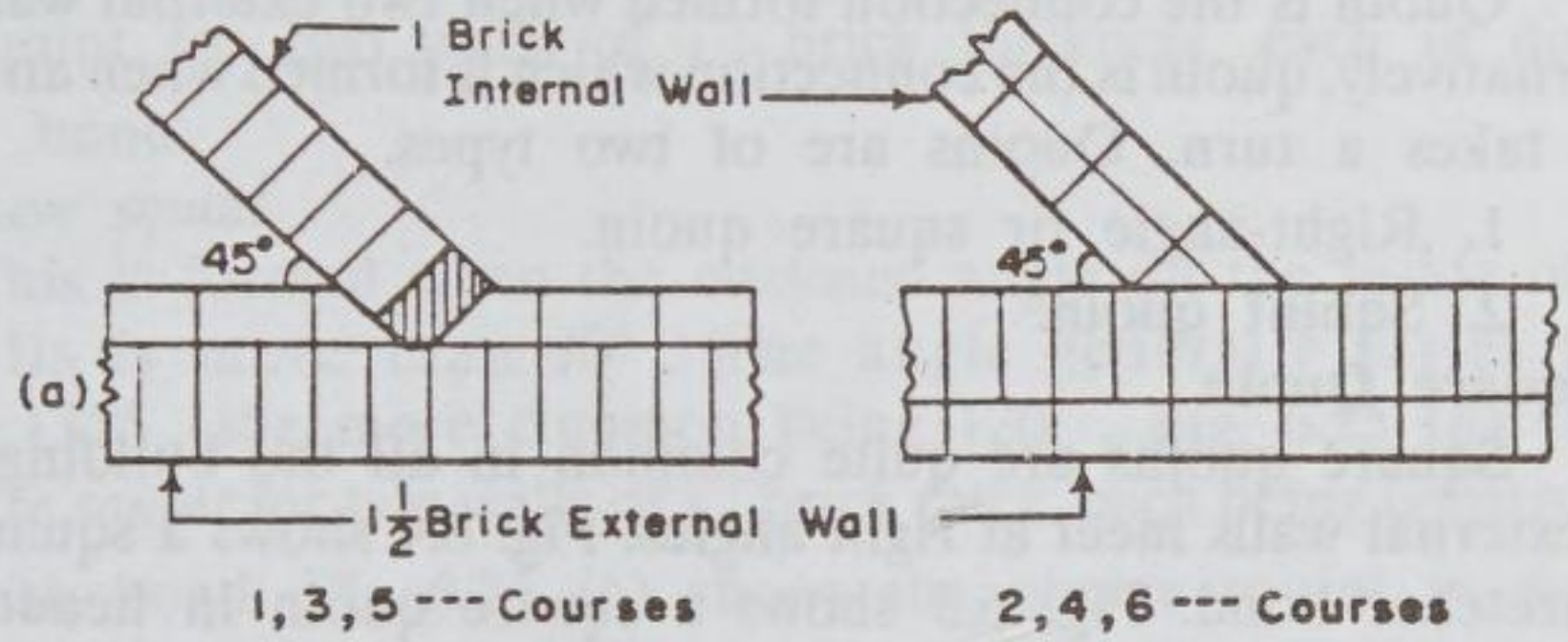
3. Squint junction

A squint junction is formed when an internal wall meets an external continuous wall at an angle other than 90°. Usually, the angle of squint is kept at 45°, though squint junctions are not very common in brick work.

(a) Squint junction in English bond

Fig. 6.22 (*a*) shows a squint junction between a $1\frac{1}{2}$-brick thick external wall and a 1-brick thick internal wall, both being constructed in English bond. The header couses of the cross-wall is taken inside the main wall, thus getting the required bond. Alternate courses of both the walls remain unbonded.

Fig. 6.22(*b*) shows a squint junction between two walls each of $1\frac{1}{2}$-brick thickness and constructed in English both. The header bricks are taken inside the main wall. Alternate courses remain unbonded.



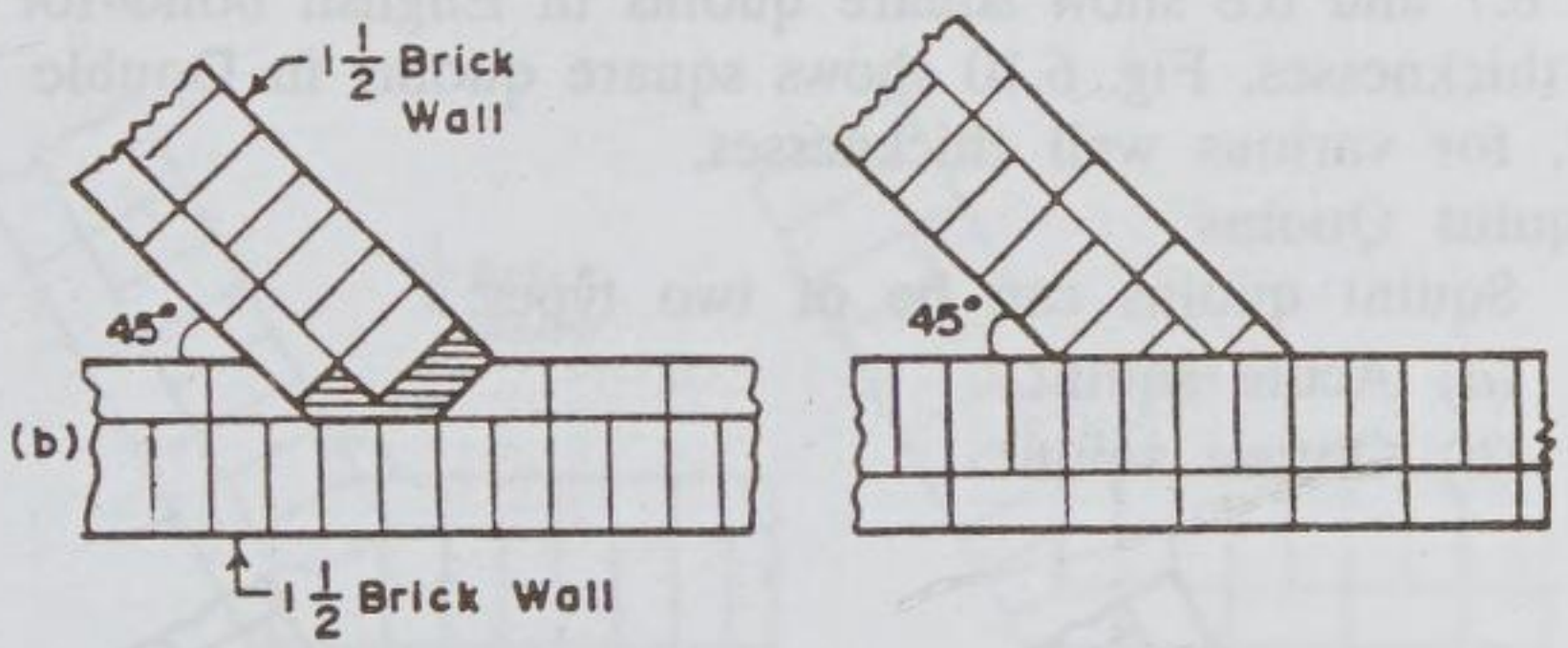
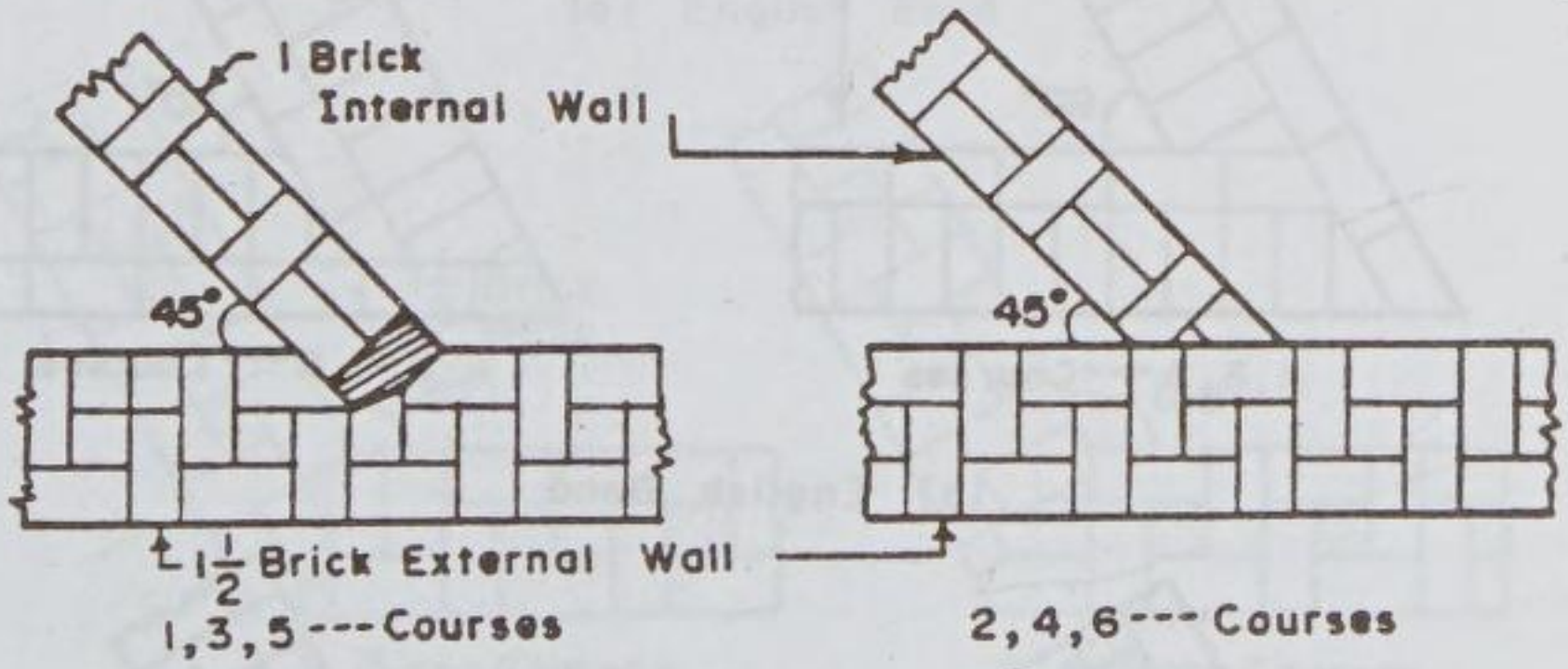


FIG. 6.22. SQUINT JUNCTION IN ENGLISH BOND.



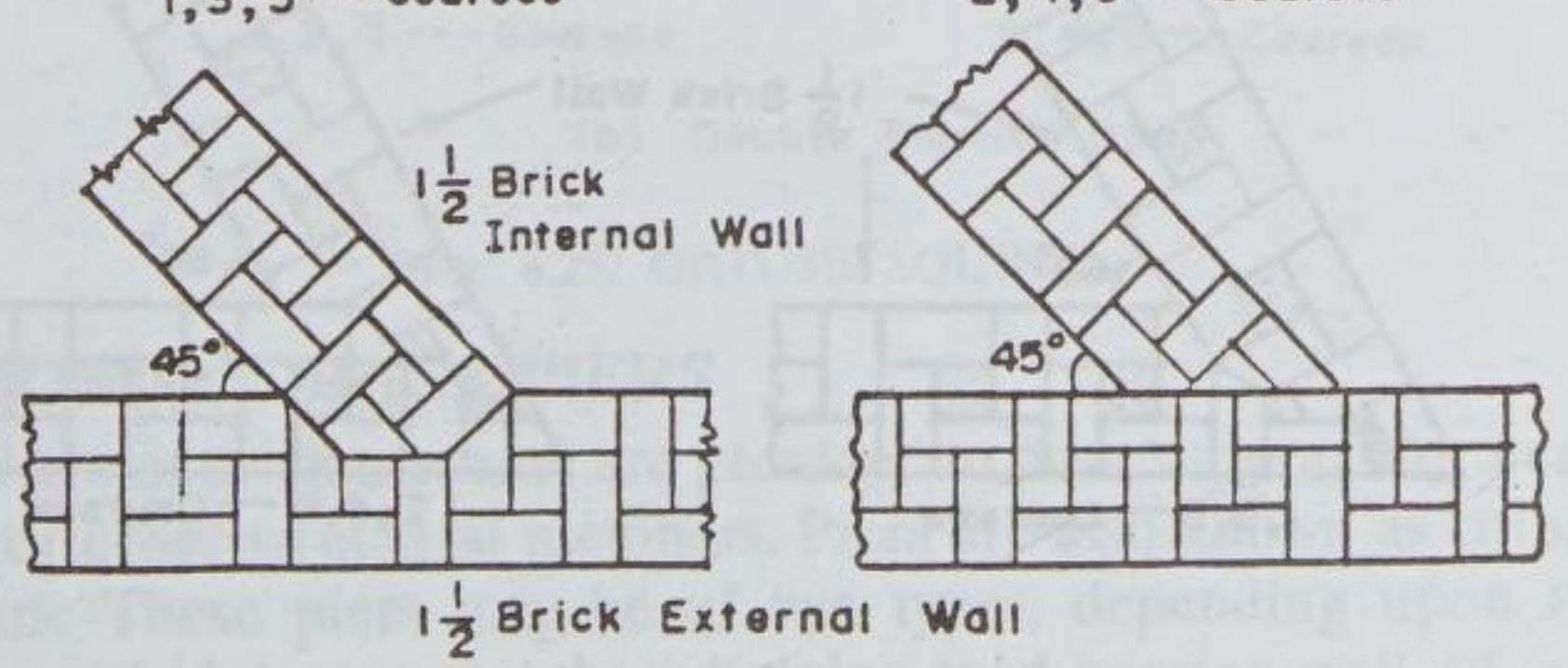


FIG. 6.23. SQUINT JUNCTION IN DOUBLE FLEMISH BONDS.

(b) Squint junction in Double Flemish bond

Fig. 6.23 shows the squint junction for the walls constructed in Double Flemish bond. These junctions are quite difficult to be constructed.

(B) QUOINS

Quoin is the connection formed when two external walls meet. Alternatively, quoin is the connection which is formed when an external wall takes a turn. Quoins are of two types.

1. Right-angle or square quoin.
2. Squint quoin.

1. Square Quoin

Square quoins are quite common in all the buildings where the external walls meet at right angles. Fig. 6.4 shows a square quoin in stretcher bond. Fig. 6.5 shows a square quoin in header bond. Figs. 6.7 and 6.8 show square quoins in English bond for various wall thicknesses. Fig. 6.10 shows square quoins in Double Flemish bond, for various wall thicknesses.

2. Squint Quoins

Squint quoins can be of two types:

(*a*) Acute squint.

(*b*) Obtuse squint.

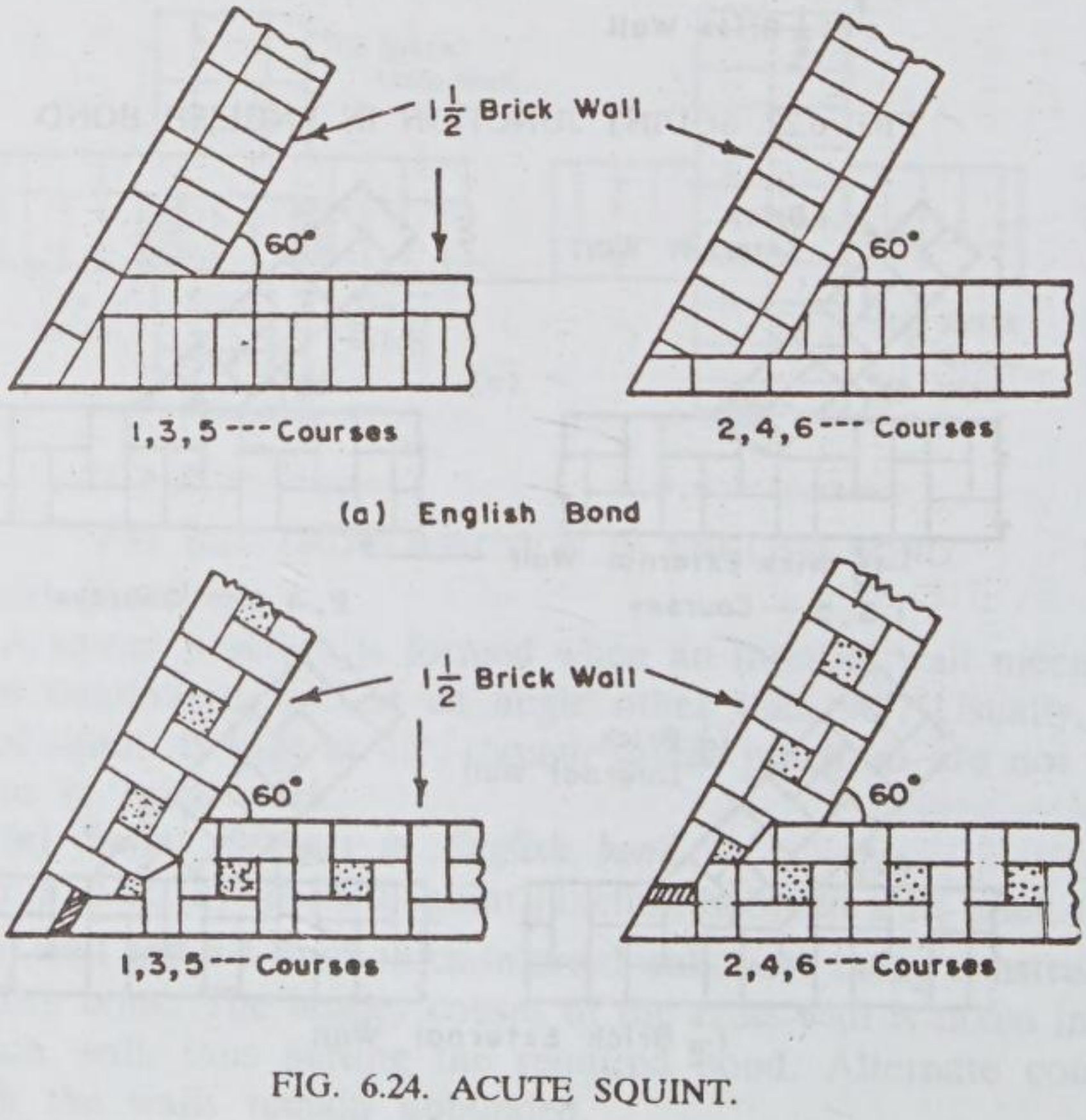


FIG. 6.24. ACUTE SQUINT.

(a) Acute squint

This is formed when the enclosed angle on the inside of the two walls is less than 90° . Generally, the acute angle is kept equal to 60° . Fig. 6.24 (*a*) shows an *acute squint* for two walls of $1\frac{1}{2}$-brick thick, each being constructed in English bond. Fig. 6.24 (*b*) shows acute squint for two walls of $1\frac{1}{2}$ brick thickness, each in double Flemish bond.

(b) Obtuse squint

This is formed when the enclosed angle on the inside of the two walls is more than 90° . The angle generally varies from 105° to 135° , the more common being 120° . Fig. 6.25 (*a*) shows the *obtuse squint* for two walls of $1\frac{1}{2}$ brick thick, each being constructed in English bond. Fig 6.25 (*b*) shows the obtuse squint in double flemish bond.

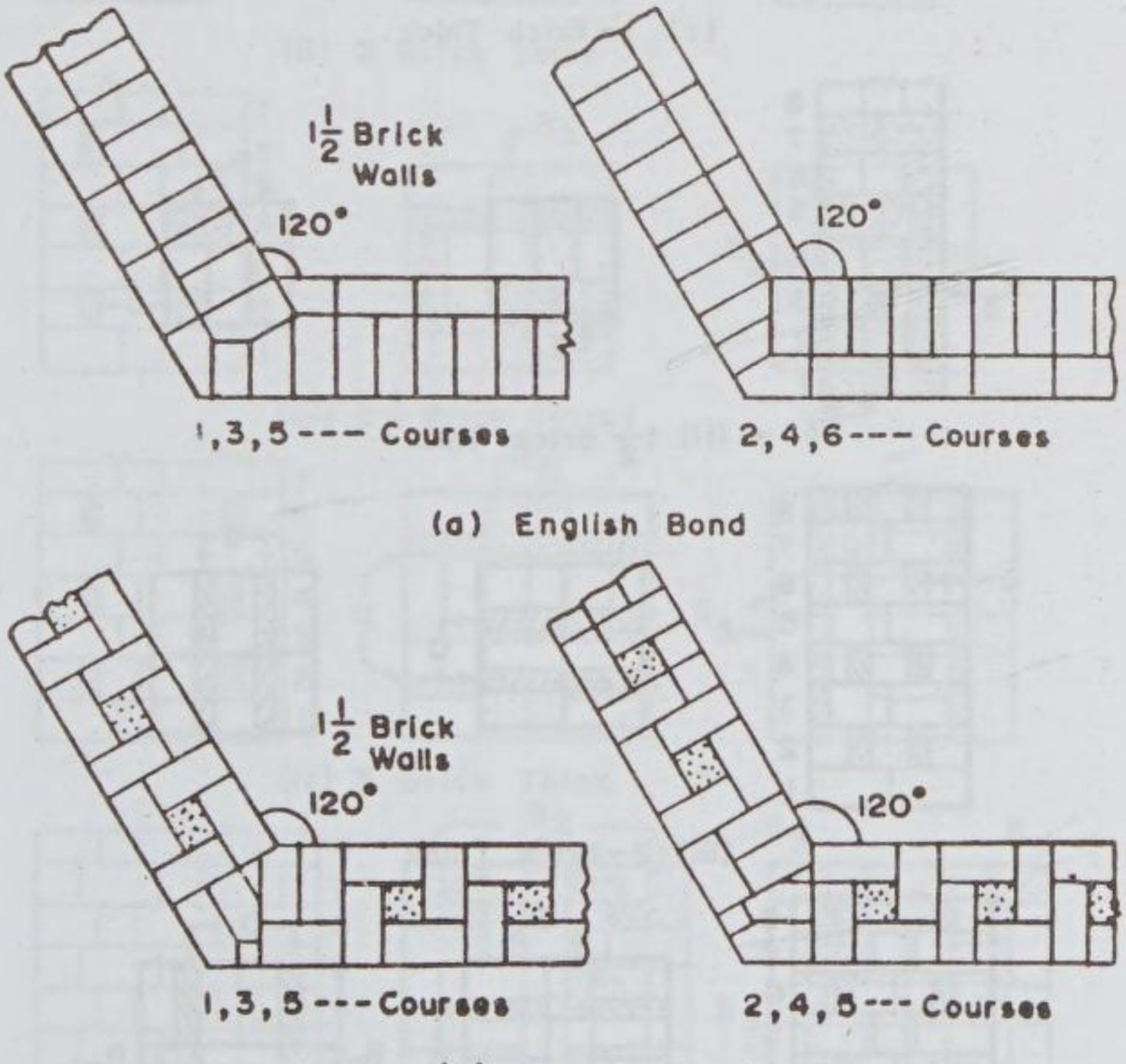


FIG. 6.25. OBTUSE SQUINT

6.17. BOND IN BRICK PIERS

Piers of brick masonry are provided to have supports for beams, trusses or other structural members. Piers are also known as columns or pillars. These piers may be of two types, depending upon their location with reference to the adjoining load bearing wall (if. any):

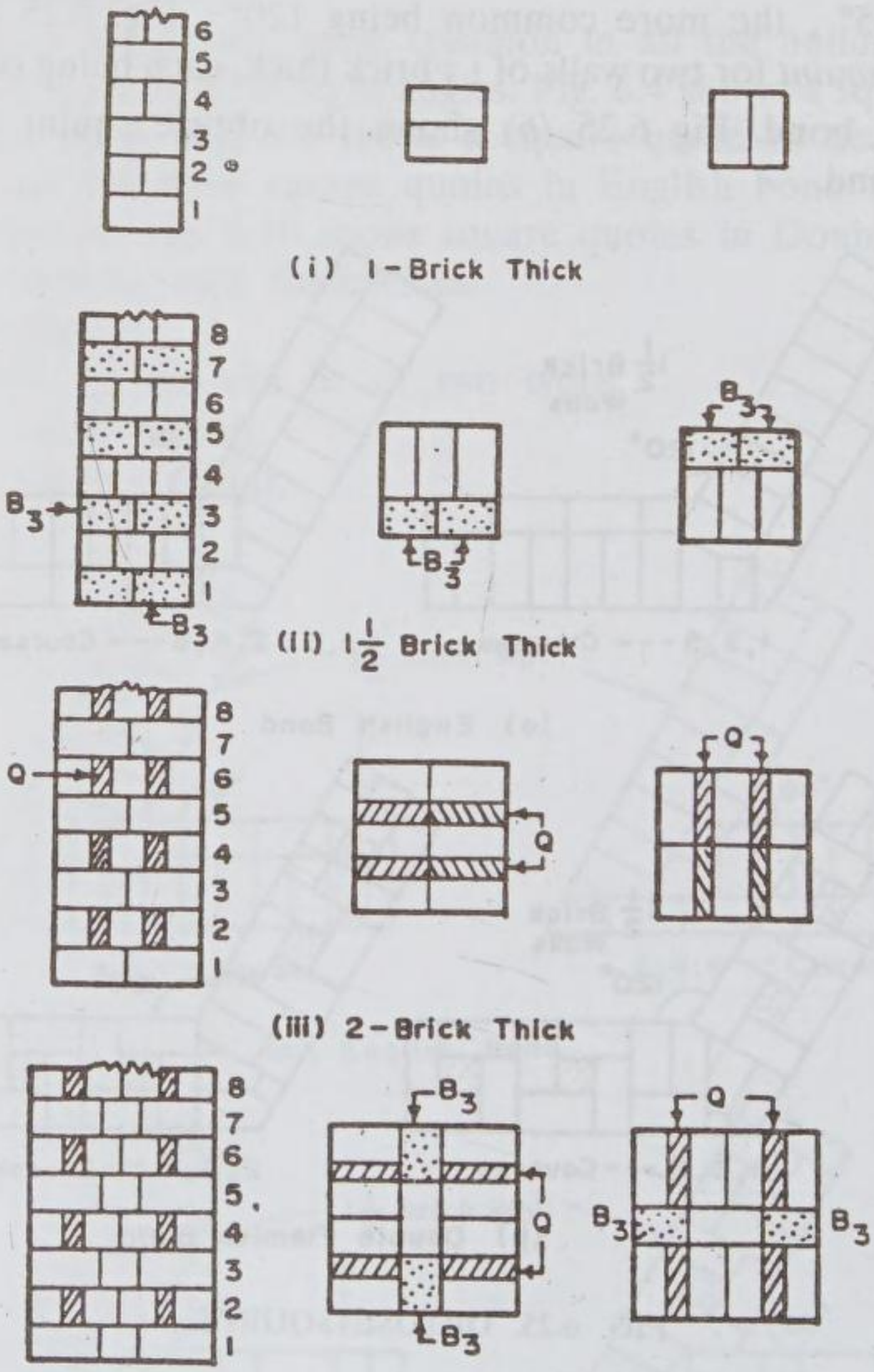
(*a*) Detached or isolated piers.
(*b*) Attached piers.

(A) ISOLATED PIERS

Though piers may be constructed in any type of bond, generally English bond or double Flemish bond is adopted. The size of the pier as well as its shape (*i.e.*, square, rectangular or circular) depends upon the magnitude of the load as well as architectural requirements.

1. Piers in English bond

Fig. 6.26 shows the piers of various thicknesses, in English bond.

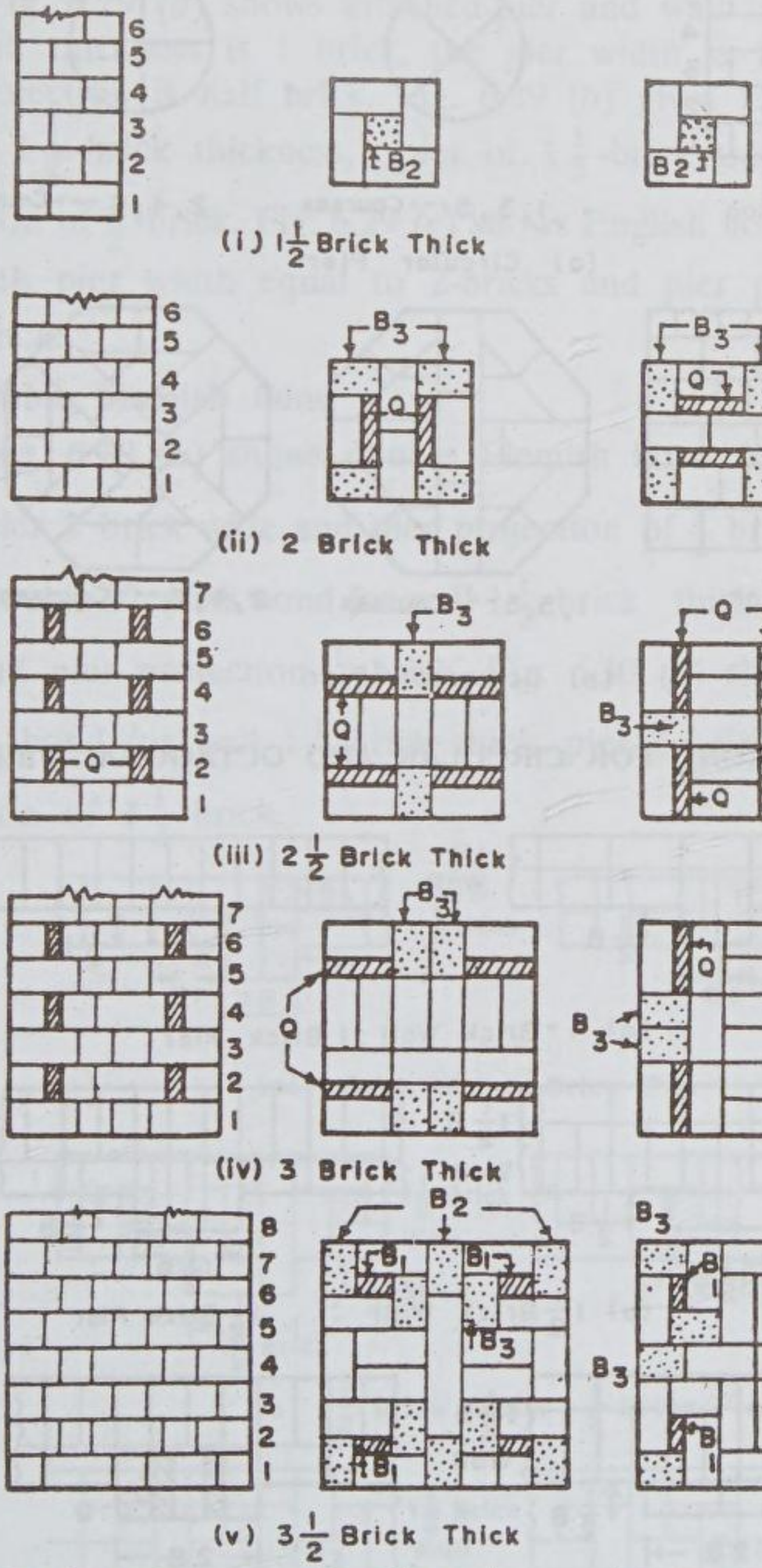


Q = QUEEN CLOSER ; B_3 = 3/4 BRICK BAT.

FIG. 6.26. PIERS IN ENGLISH BOND.

(*b*) Piers in double Flemish bond

Fig. 6.27 shows the piers of various thicknesses in double flemish bond..



Q = QUEEN'S CLOSER ; $B_3 = \frac{3}{4}$ BRICK BAT ; $B_1 = \frac{1}{4}$ BRICK BAT

FIG. 6.27. PIERS IN DOUBLE FLEMISH BOND

(*c*) Circular and Octagonal Piers

Fig. 6.28 shows bond for circular and octagonal piers.

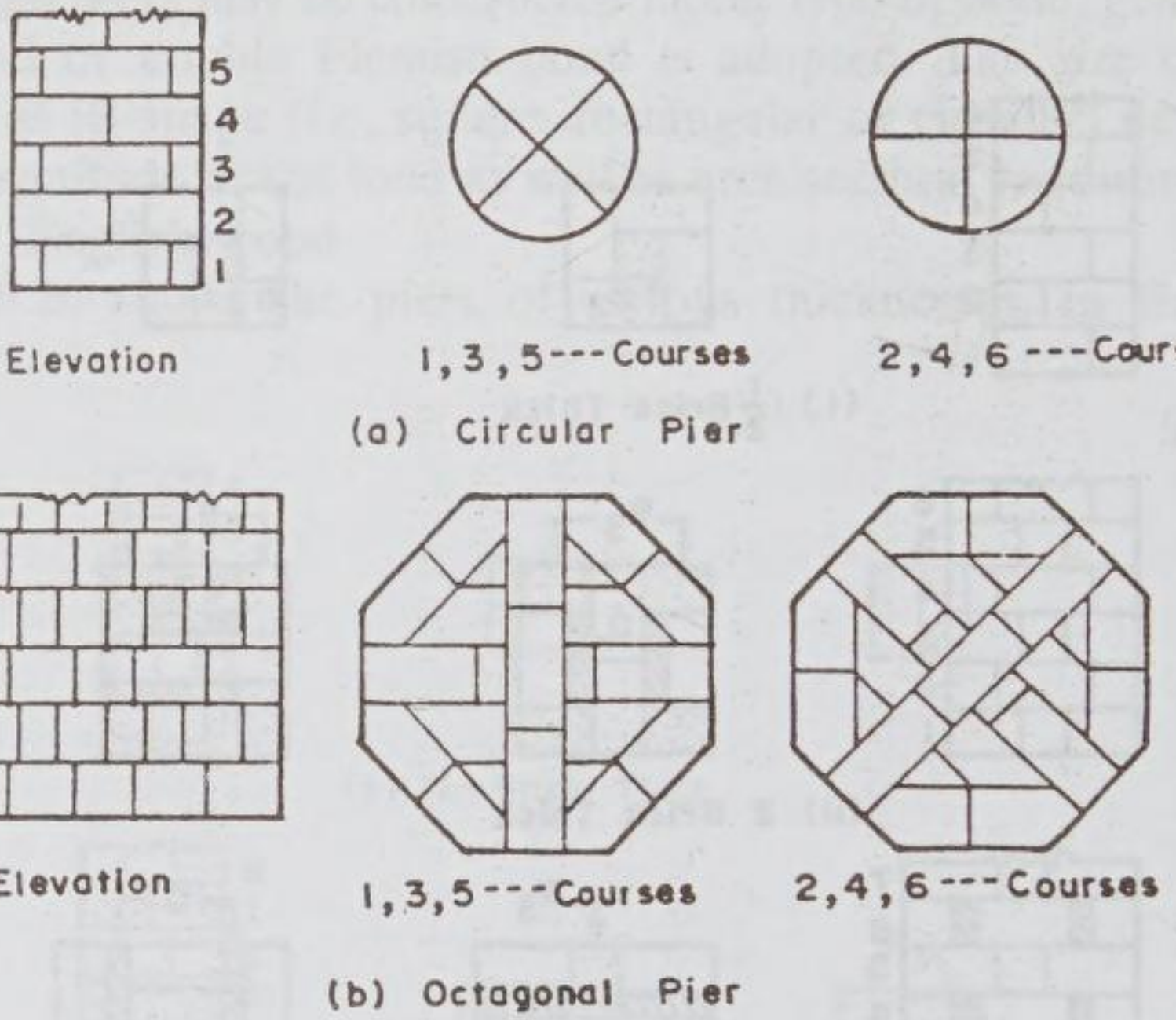


FIG. 6.28. BOND FOR CIRCULAR AND OCTAGONAL PIERS.

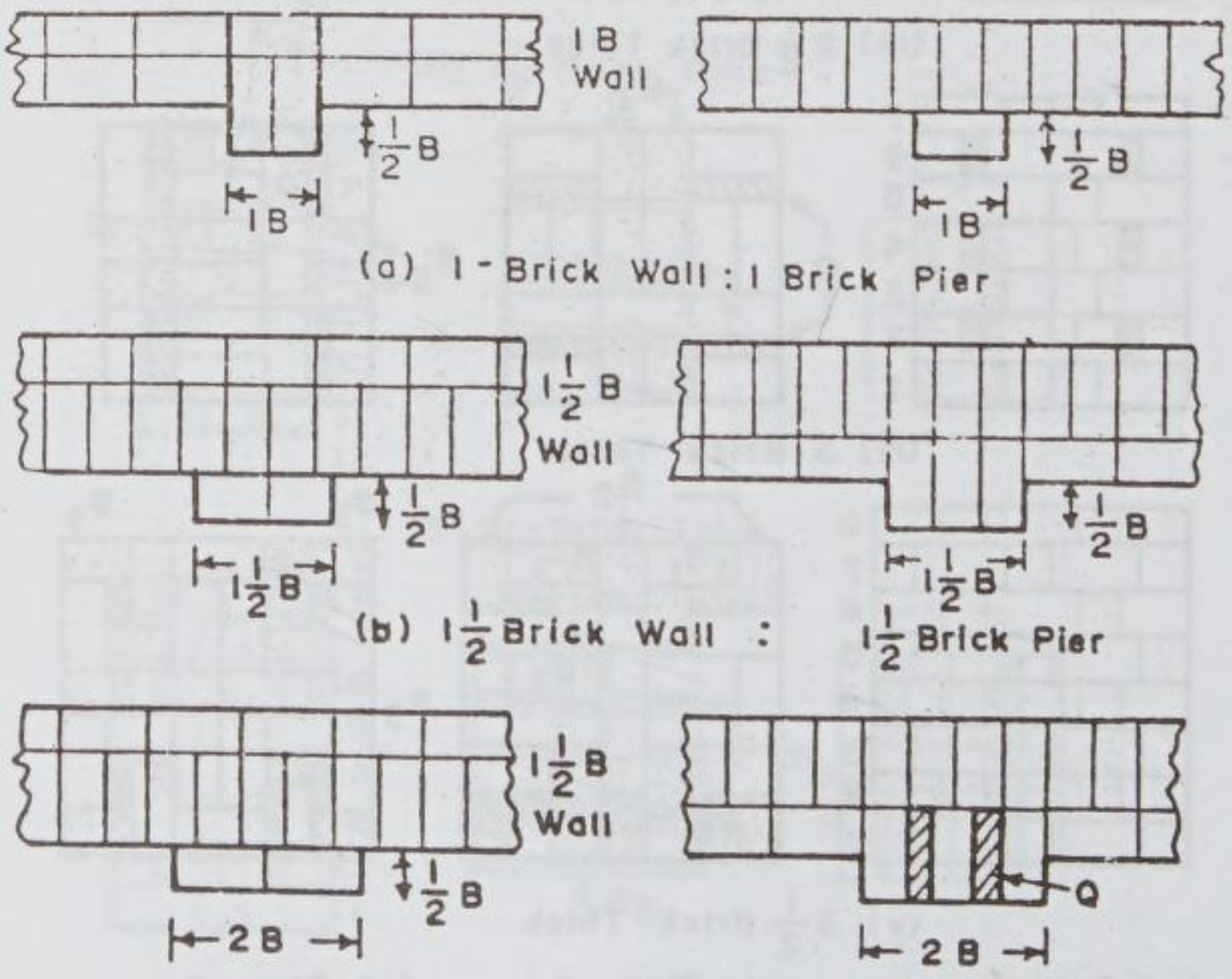

1 B Wall
1/2 B
1 B
(a) 1 - Brick Wall : 1 Brick Pier
1 1/2 B Wall
1/2 B
1 1/2 B
(b) 1 1/2 Brick Wall : 1 1/2 Brick Pier
1 1/2 B Wall
1/2 B
2 B
Q
(c) 1 — Brick Wall : 2 Brick Pier

FIG. 6.29. ATTACHED PIERS IN ENGLISH BOND.

(B) ATTACHED PIERS

Attached piers are constructed along the wall for two purposes:

(*i*) to provide larger bearing area for supporting heavy girders, roof etc. and

(*ii*) to provide stiffness to the wall.

(*a*) English bond

Fig. 6.29 (*a*) shows attached-pier and wall in English bond. The wall thickness is 1 brick, the pier width is 1 brick and the pier projection is half brick. Fig. 6.29 (*b*) gives English bond for wall of $1\frac{1}{2}$-brick thickness, pier of $1\frac{1}{2}$-brick thickness and pier projection of $\frac{1}{2}$-brick. Fig. 6.29 (*c*) shows English bond for $1\frac{1}{2}$-brick wall with pier width equal to 2-bricks and pier projection equal to $\frac{1}{2}$-brick.

(*b*) Double Flemish bond

Fig. 6.30 (*a*) shows double Flemish bond for wall $1\frac{1}{2}$-brick thick, pier 1 brick wide and pier projection of $\frac{1}{2}$ brick. Fig. 6.30(*b*) shows double Flemish bond for wall $1\frac{1}{2}$-brick thick, pier $1\frac{1}{2}$-brick wide and pier projection $\frac{1}{2}$-brick. Fig. 6.30 (*c*) shows the double Flemish bond for wall $1\frac{1}{2}$-brick thick, pier 2 brick wide and pier projection of $1\frac{1}{2}$ brick.

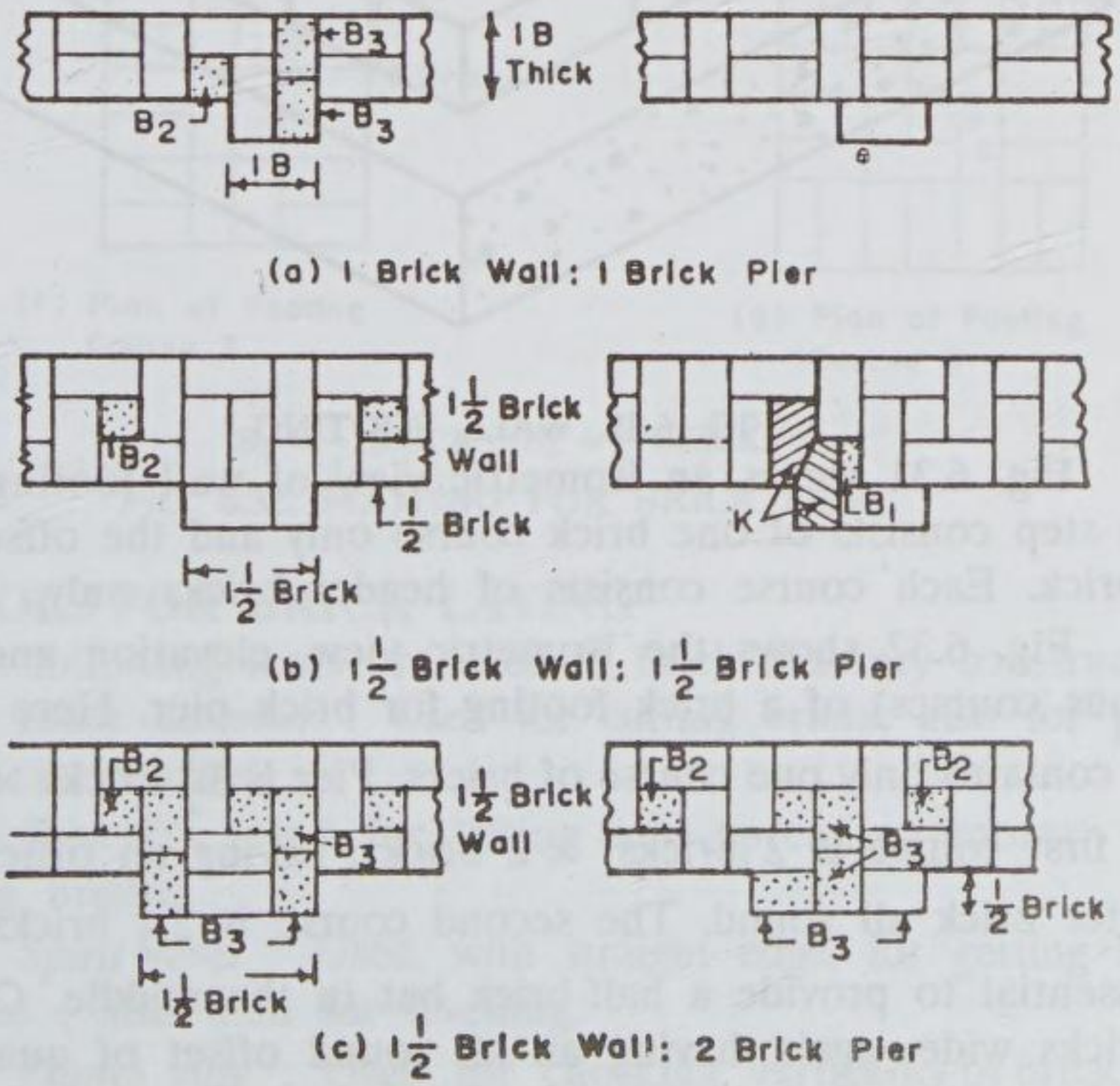


FIG. 6.30. ATTACHED PIERS IN FLEMISH BOND.

K= KING CLOSER ; $B_2=\frac{1}{2}$ BRICK BAT ; $B_1=\frac{1}{4}$ BRICK BAT ;

6.18. BOND IN FOOTINGS

Footings distribute the load of wall or pier, to a wider area at its base, through the provision of steps or offsets. Each step of the footing can be constructed either in single course of bricks or in double or more courses. Footings of single course of brick in each step is adopted for light loads. In such a case the bricks are laid as headers on the outside. This would make it possible to give greater bearing to the projecting portion (offset) inside the wall or pier. In the case of double or multiple courses, the method of construction and bonding is similar to that adopted for the wall or pier.

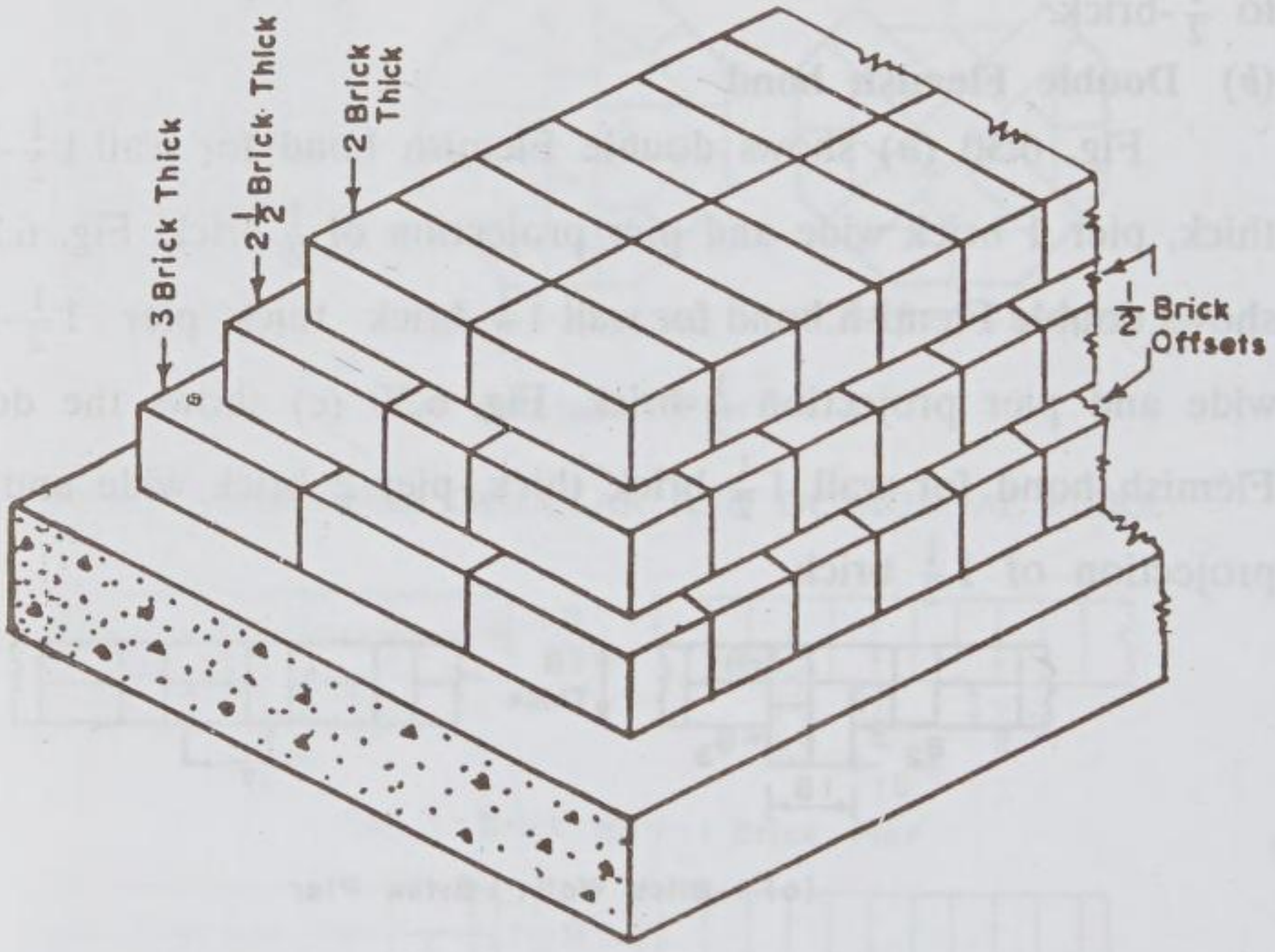
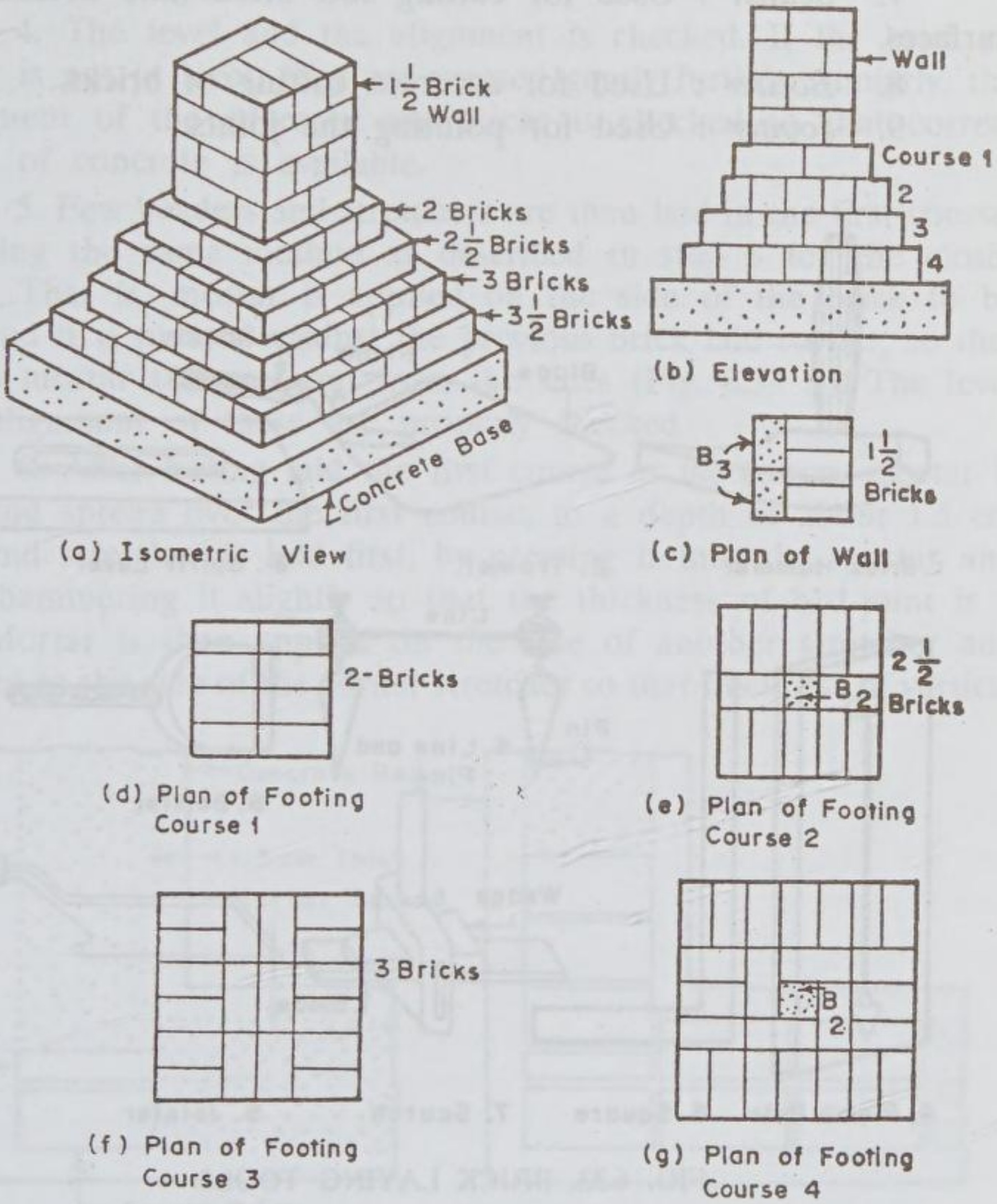


FIG. 6.31. WALL FOOTING.

Fig. 6.31 shows an isometric view of wall footing in which each step consists of one brick course only and the offset is equal to brick. Each course consists of header bricks only.

Fig. 6.32 shows the isometric view, elevation and plan (of various courses) of a brick footing for brick pier. Here also, each step contains only one course of bricks. Pier is $1\frac{1}{2}$ bricks $\times$ $1\frac{1}{2}$ bricks. The first course is 2 bricks $\times$ 2 bricks, having an offset equal to quarter brick all round. The second course is $2\frac{1}{2}$ bricks wide ; it is essential to provide a half-brick bat in the middle. Course 3 is 3 bricks wide, again having an all round offset of quarter brick. In this course all the bricks are full bricks. The fourth course of

footing is $3\frac{1}{2}$ bricks wide. It has a Flemish bond pattern at its middle, with a half-brick bat. Remaining all bricks are full bricks.



$B_3 = \frac{3}{4}$ BRICK ; $B_2 = \frac{1}{2}$ BRICK

FIG. 6.32. FOOTING FOR BRICK PIER.

6.19. TOOLS FOR BRICK LAYING

The following tools are used in brick masonry construction.

1. *Brick hammer* : Used for cutting bricks; also for pushing the bricks in courses.

2. *Trowel* : Used for lifting and spreading mortar ; also for cutting bricks.

3. *Spirit level* : Used, with straight edge, for getting horizontal surface ; also used for levelling.

4. *Plumb rule* : Used for checking verticality of brick walls.

5. *Mason's square* : Used for checking right angles.
6. *Line and pins* : Used for maintaining alignment of courses.
7. *Scutch* : Used for cutting soft bricks and dressing out surfaces.
8. *Bolster* : Used for accurate cutting of bricks.
9. *Jointer* : Used for pointing the joints.

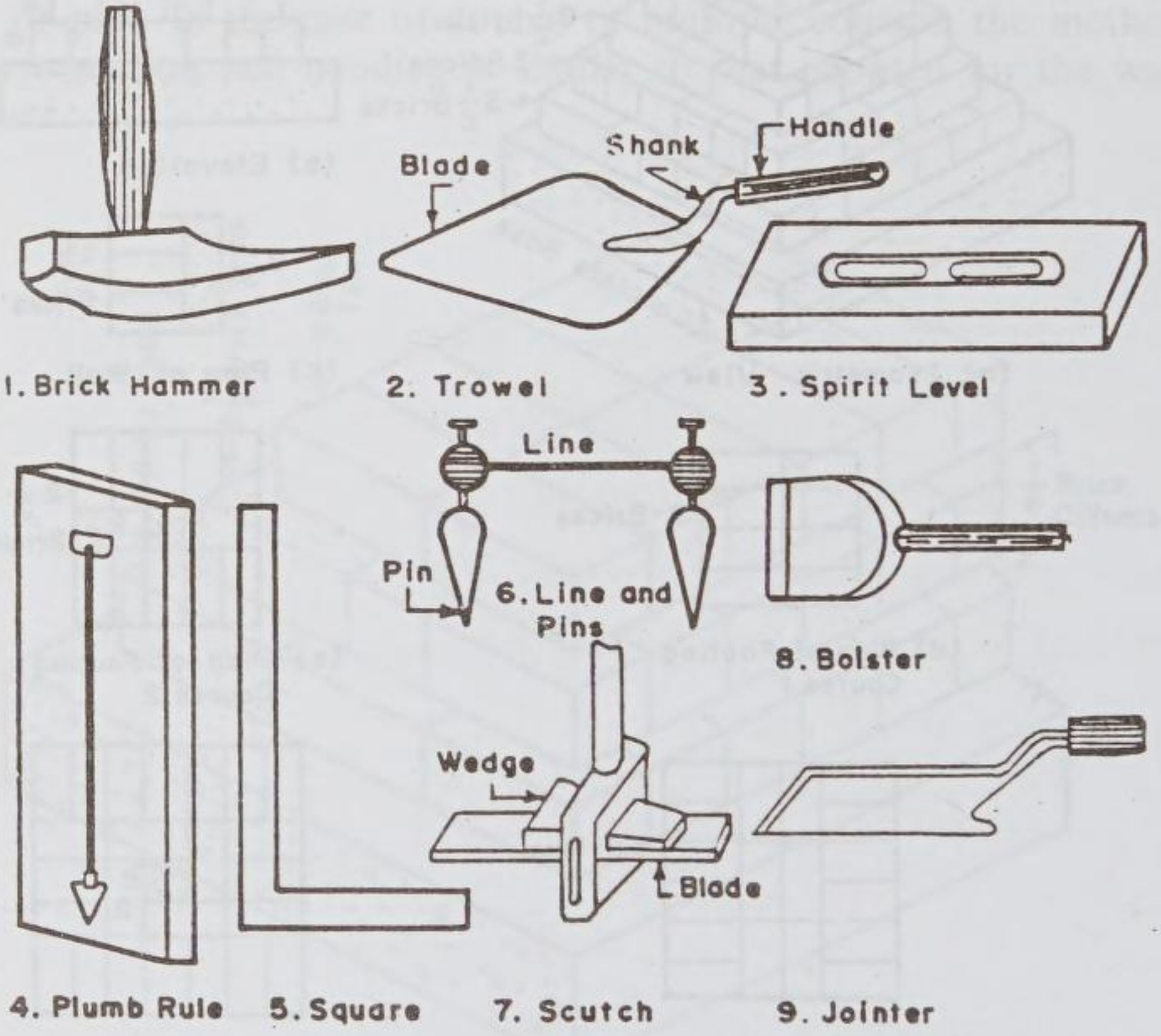


FIG. 6.33. BRICK LAYING TOOLS.

6.20. BRICK LAYING

Brick masonry construction is a great art since laying must be systematically done with respect to bonding, jointing and finishing. Brick laying for wall construction is done in the following steps :

1. All the bricks to be used in construction are thoroughly soaked in water so that they do not absorb the water of the mortar.
2. Mortar is spread on the top of the foundations course, over an area to be covered by the edges of the wall. The depth of spread of mortar may be about 1.5 cm.
3. The corner of the wall is constructed first. For that, one brick is laid first at the corner and pressed with hand so that the thickness of bed-joint remains only about 1 cm. The *first closer* is covered with mortar on its side and then pressed against the first

corner brick, such that 1 cm thick vertical joint is obtained. The excess mortar from the sides will squeeze out, which is cleaned off with trowel (Fig. 6.34 *a*).

4. The level and the alignment is checked. If the brick or closer is not in level, they are pressed gently further. Simiarly, the placement of the edges of the bricks is checked so that correct offset of concrete is available.

5. Few headers and stretchers are then laid in the first course, adopting the same method as described in step 3 for the closer brick. That is, mortar is applied on the side of the brick to be laid and it is pressed against the previous brick laid earlier, so that excess mortar squeezes out from the sides (Fig. 6.34 *b*). The level and alignment of these are properly checked.

6. After having laid the first course *at the corner*, mortar is laid and spread over the first course, to a depth of about 1.5 cm and end stretcher is laid first, by pressing it into the mortar and then hammering it slightly so that the thickness of bed-joint is 1 cm. Mortar is then applied on the side of another stretcher and pressed to the side of the corner stretcher so that thickness of vertical

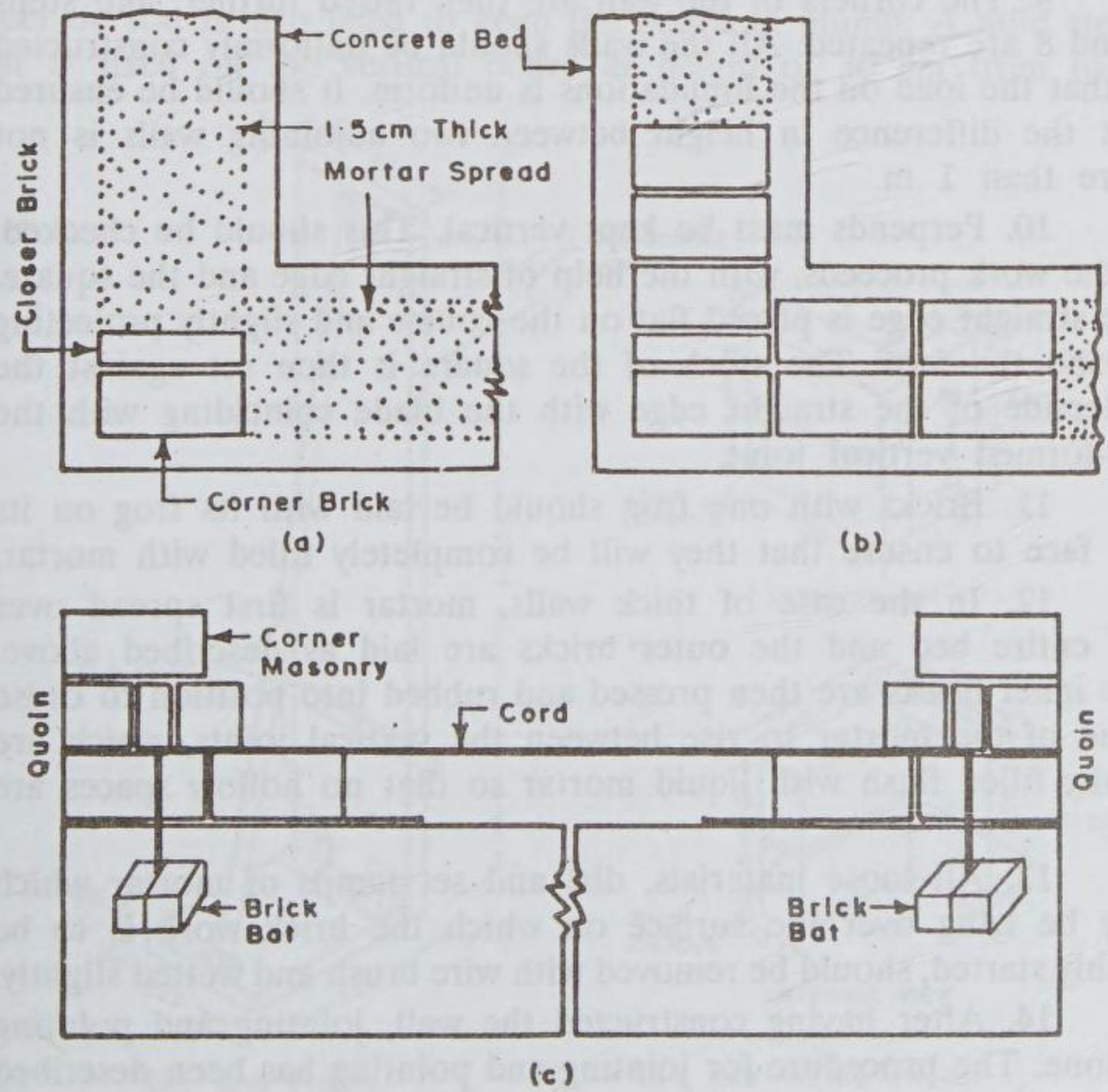


FIG. 6.34. BRICK LAYING BY CONVENTIONAL METHOD.

joint is about 1 cm. Excess mortar which oozes out is cleaned off. This way, stretchers and headers are laid for the second course.

7. Other courses (usually four to six) are then laid at the corner. Similarly, the corner at the other end of the wall is laid. Since the corner construction at each end works as a guide for filling in-between bricks of various courses, the corner construction should be done with great care. The plumb as well as alignment should be thoroughly checked. *Plumbing up* by means of plumb rule should be frequently resorted to as new brick work has a tendency to overhang. Vertical face is obtained by tapping the handle of the trowel against the overhanging bricks.

8. For building the in-between portion of the wall, a cord is stretched along the top of the first course laid at each corner, as shown in Fig. 6.34 (*c*). A brick bat is attached at either end of the cord so that it remains tout. The course is then built. The line or cord is then shifted up, corresponding to the top level of the second course, and the second course is also constructed. The procedure is repeated till the in-between wall is constructed to the height of corner masonry.

9. The corners of the wall are then raised further, and steps 7 and 8 are repeated. All the walls should be uniformly constructed so that the load on the foundations is uniform. It should be ensured that the difference in height between two adjoining walls is not more than 1 m.

10. Perpends must be kept vertical. This should be checked, as the work proceeds, with the help of straight edge and the square. The straight edge is placed flat on the course and slightly projecting beyond the face. The stock of the square is then set against the underside of the straight edge with the blade coinciding with the last-formed vertical joint.

11. Bricks with one frog should be laid with its frog on its top face to ensure that they will be completely filled with mortar.

12. In the case of thick walls, mortar is first spread over the entire bed and the outer bricks are laid as described above. The inner bricks are then pressed and rubbed into position to cause some of the mortar to rise between the vertical joints, which are finally filled flush with liquid mortar so that no hollow spaces are left.

13. All loose materials, dirt and set lumps of mortar which may be lying over the surface on which the brick work is to be freshly started, should be removed with wire brush and wetted slightly.

14. After having constructed the wall, jointing and pointing is done. The procedure for jointing and pointing has been described separately. However, all the joints should be cleaned and finished

after every day's work.

6.21. IMPROVED METHOD OF BRICK-LAYING

An improved method of brick laying has been developed by CBRI, wherein delays are eliminated by well-organised work place layout using *new gadgets* consisting of (*i*) *end frame* (*ii*) *string holder*, and (*iii*) *mortar board*, and arranging the brick layers in sequence of hand operation to give a rhythm to the movement pattern. The method recommended in the *Handbook of Building Construction Practices* (Indian Standard Institution), is reproduced here.

1. Special Gadgets

(*i*) *End Frames*

An end frame shown in Fig. 6.35 (*a*) is made of 25 mm thick hard board or timber and the height is generally kept 1.25 m. In case well-seasoned timber is not available, this can be made by making cored frame work of timber and fixing 5 mm thick shuttering plywood or hard boards on both sides. At the sides, top and bottom timber lapping is provided. The width of the board is kept equal to the thickness of the wall. The vertical board and base board are jointed at right angles by two pieces of angle iron, and a mild steel tie rod is also fixed to keep the board in plumb. A mild steel flat is fixed on the vertical board at height of 30 cm from base

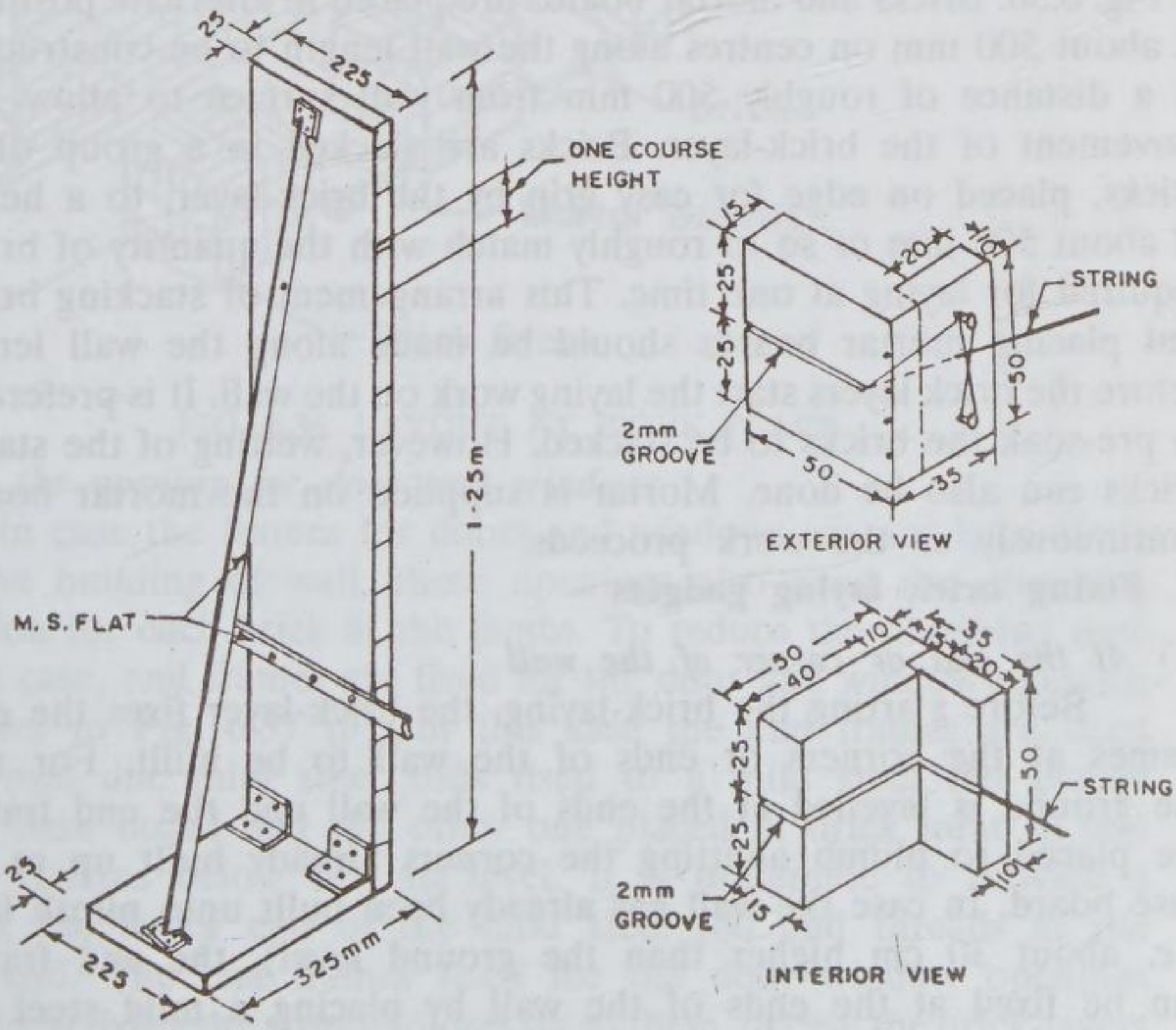


FIG. 6.35. SPECIAL GADGETS.

board for fixing the end frame on to the wall. Depending upon the average thickness of bricks and the horizontal mortar joint, marks as the course levels are made on both sides of the vertical board along its thickness.

(ii) String-holder

A string-holder, shown in Fig. 6.35 (*b*) is made of hard board or timber in the form of L–shape. It is 5 cm high and the lengths of the two flanges are 50 mm and 350 mm. The shorter flange has 1 mm deep groove in the centre on the inner side to position the thread and on the outer face it has two wood screws kept projecting out by about 5 mm to which the brick layer's thread is tied and kept hanging. The longer flange has a through groove or slit, 2 mm wide and 40 mm long, in the centre to allow the thread to be passed through it.

(iii) Mortar board

For keeping the mortar near the brick wall, hard boards of 500 × 500 × 25mm are used in place of conventional metal pans and these are placed on bricks to keep them at a higher level. This eliminates the interference in brick layer's hand motion due to the sides of the metal pan.

2. Layout of the work place

The general layout arrangement of the work place is shown in Fig. 6.36. Bricks and mortar boards are placed in alternate positions at about 500 mm on centres along the wall length to be constructed, at a distance of roughly 500 mm from wall surface to allow free movement of the brick-layer. Bricks are stacked in a group of 12 bricks, placed on edge for easy grip by the brick-layer, to a height of about 500 mm or so to roughly match with the quantity of bricks required for laying at one time. This arrangement of stacking bricks and placing mortar boards should be made along the wall length before the brick layers start the laying work on the wall. It is preferable to pre-soak the bricks to be stacked. However, wetting of the staked bricks can also be done. Mortar is supplied on the mortar boards continuously as the work proceeds.

3. Fixing brick laying gadgets

(i) At the end or corner of the wall

Before s arting the brick-laying, the brick-layer fixes the end-frames at the corners or ends of the wall to be built. For this, the ground is levelled at the ends of the wall and the end frames are placed to plumb abutting the corners already built up at the base board. In case the wall has already been built upto plinth level (*i.e.* about 30 cm higher than the ground level), the end frames can be fixed at the ends of the wall by placing a mild steel flat along the vertical joint of the upper course at a distance about

750 mm from the end of corner and inserting 10 mm mild steel rods threaded at both ends into the grooves on the mild steel flats in and on the frame and tightening them by butterfly nuts. Having fixed the end-frames, a string-holder, having brick-layers thread mainly passing through the slit and part of thread tied to the wood screws is positioned on the end-frame as fair face of the wall at appropriate course level. The thread is kept pulled and the other end of the thread is passed through the slit of second string-holder. Keeping the length of thread equal to the wall, the thread is tied to the wood screws of the string-holder and it is positioned at appropriate level of the end frame.

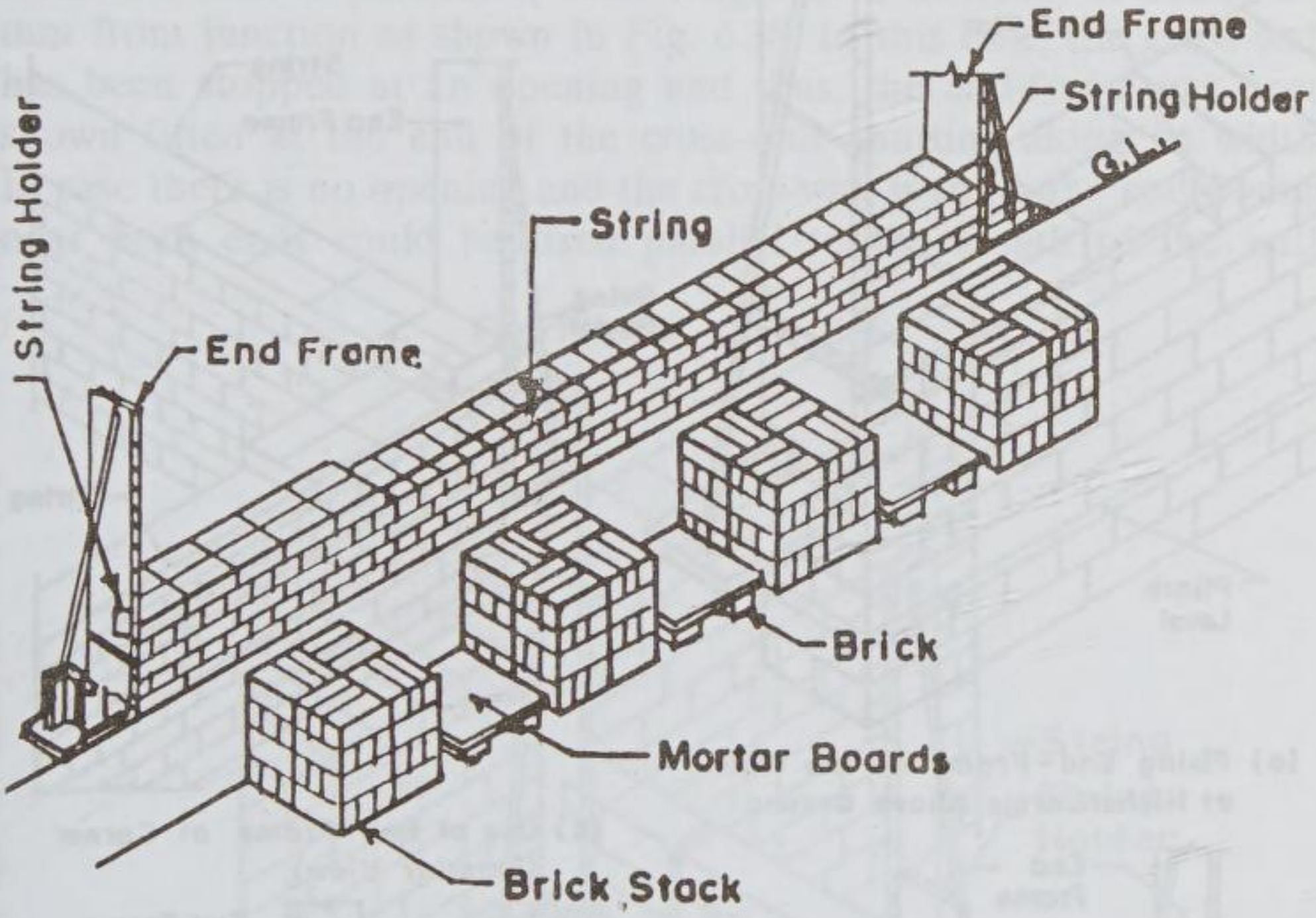


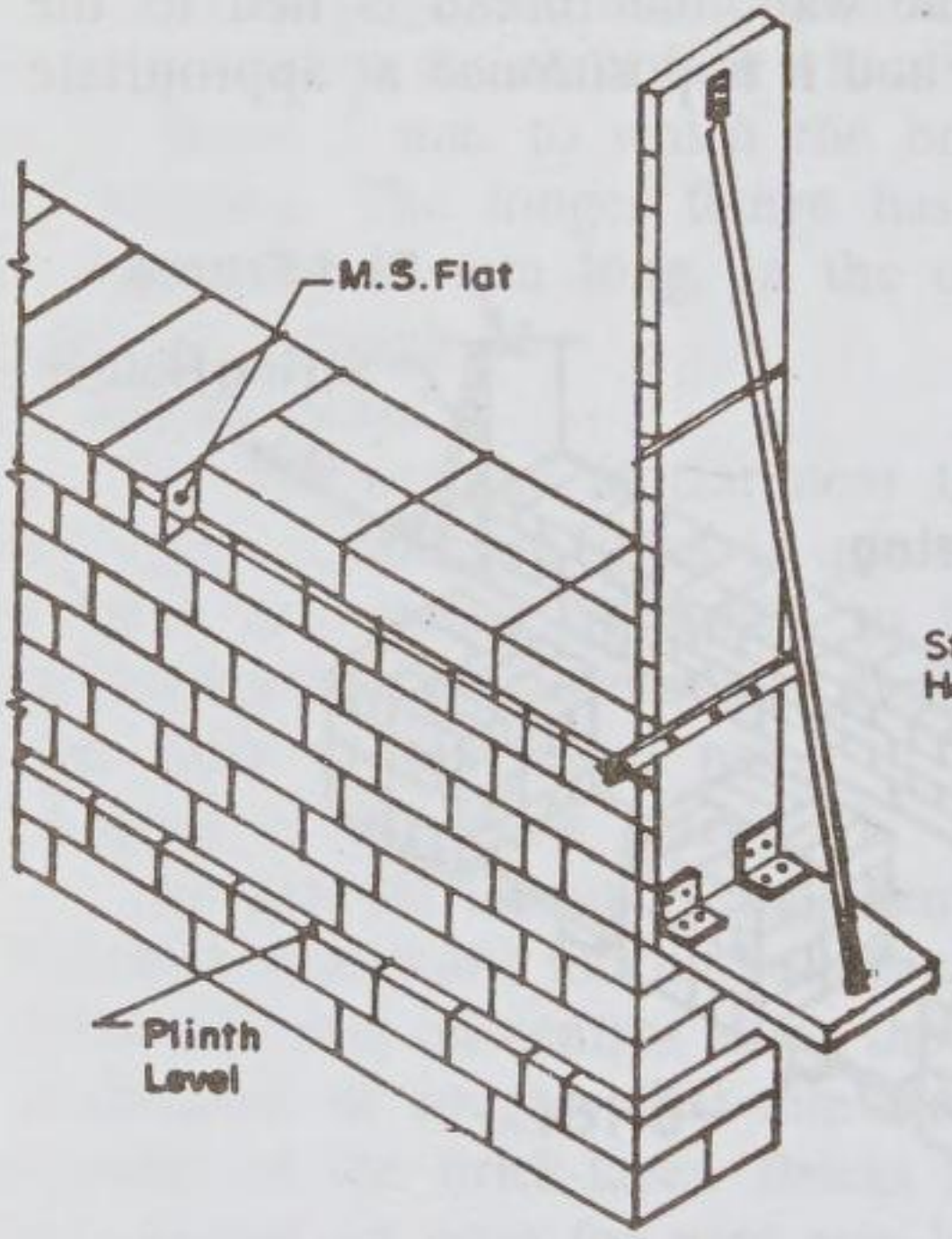
FIG. 6.36. LAYOUT AT WORK PLACE.

(b) At the opening for door and windows

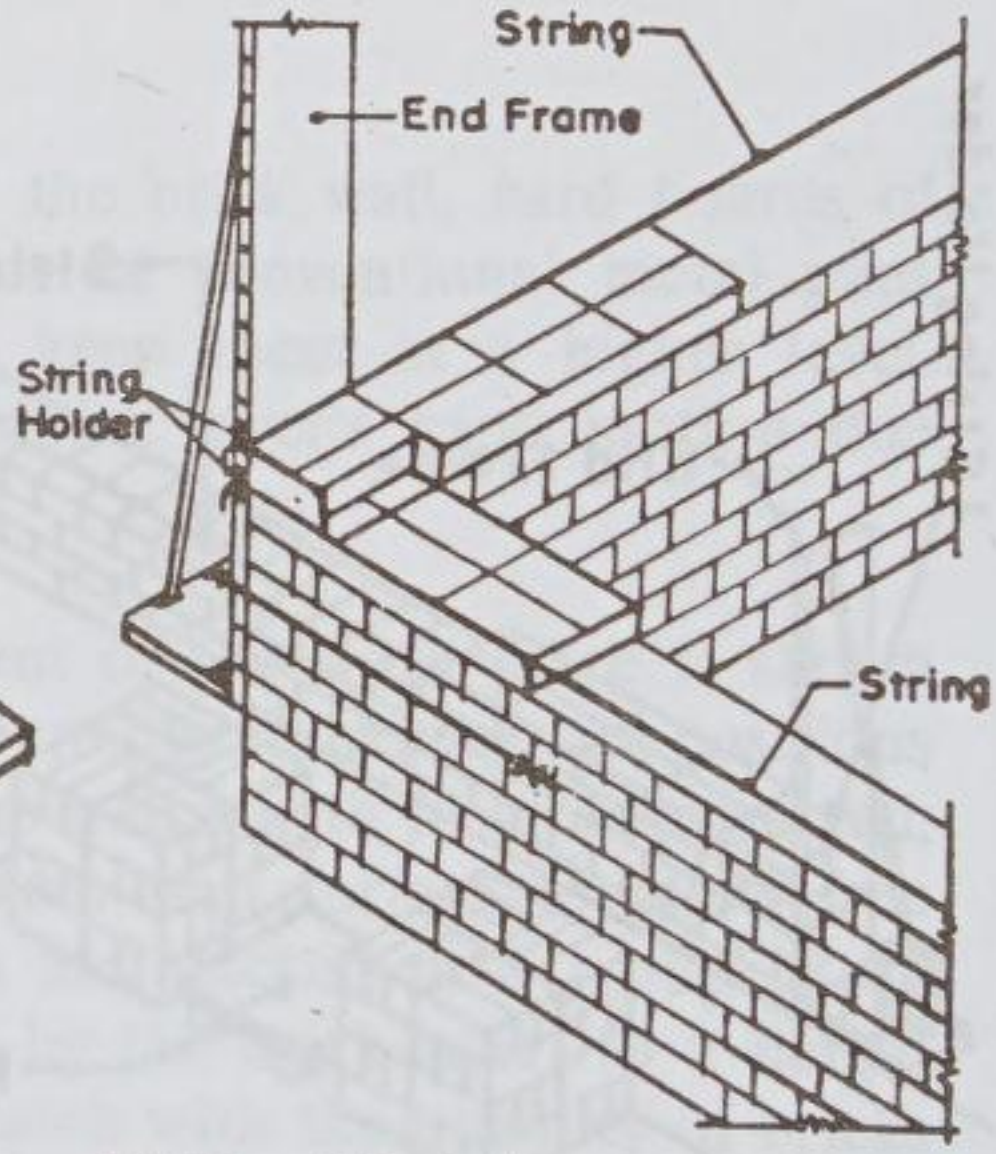
In case the frames for doors and windows are not kept along with the building of wall, these openings also need the plumbing operation for each brick at the jambs. To reduce the plumbing time in such case, end frames are fixed for the door and window opening, as shown in Fig. 6.37 (*b*). In this case the end frames are fixed by 10 mm dia. mild steel rods fixed to a mild steel flat placed on the base board and the other one placed in brick joint in one of the courses below the sill level. It is preferable to provide a loop at the lower end of the mild steel rod and threads at the upper end. The end frames fixed for the door window openings also help in fixing the string-holders on to them, in case the brick-layers build the wall in part lengths.

(c) At corner for building cross walls simultaneously

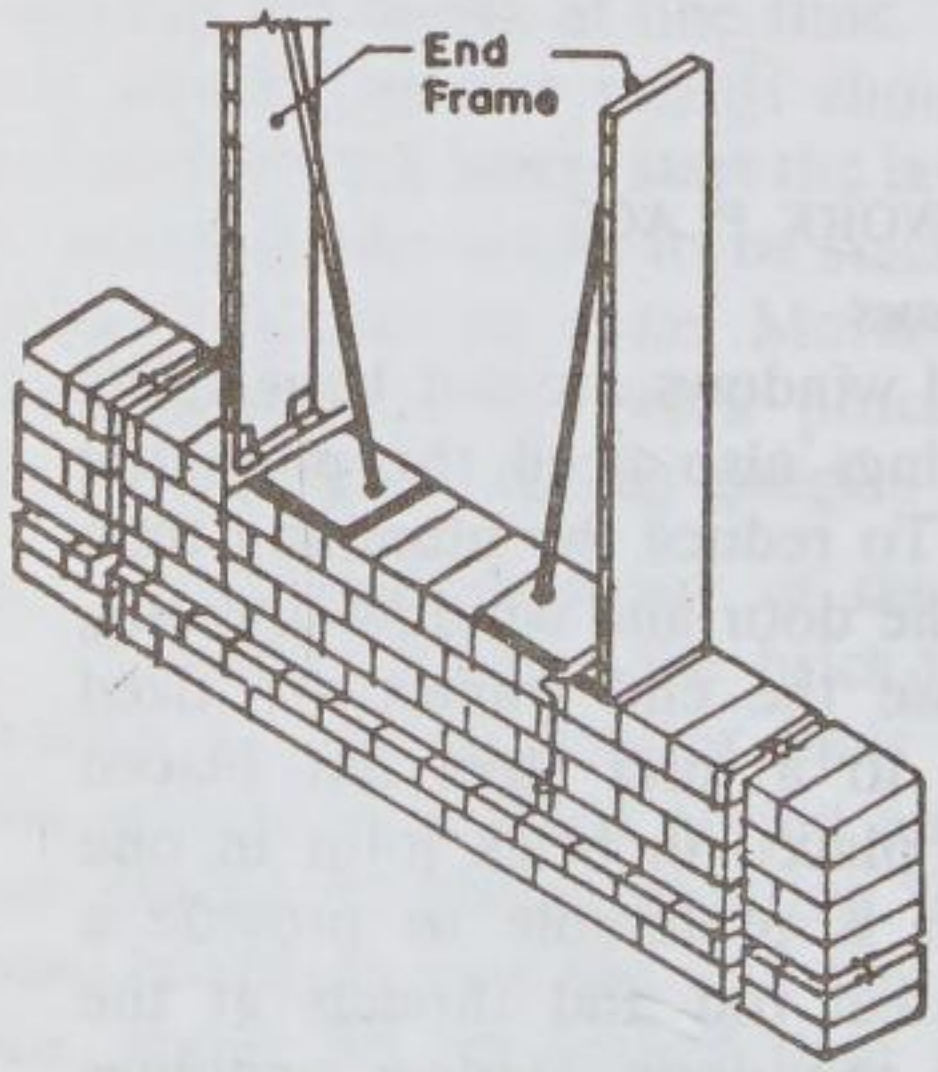
At buildings sites, often a gang of brick layers work and more than one walls are built simultaneously. When two walls at a corner are built simultaneously, it is possible to build them using only one end frame at the corner as shown in Fig. 6.37 (*c*) and (*d*). The important consideration is that the two should not be built at the same course level at a time but the wall along the end of which



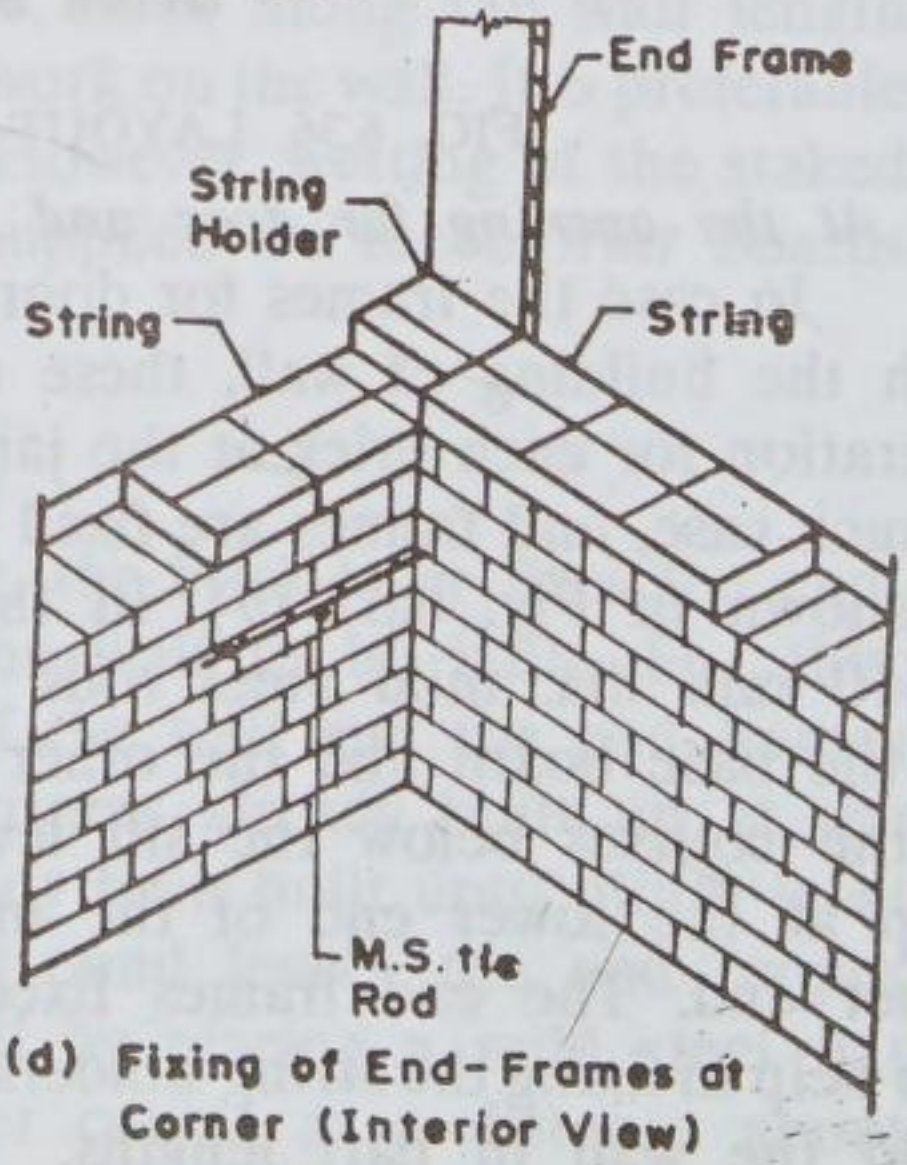
(a) Fixing End-Frame on the Wall at HigherLevels Above Ground



(c) Use of End-Frame at Corner (Exterior View)



(b) Fixing End-Frames for Window or Door Openings



(d) Fixing of End-Frames at Corner (Interior View)

FIG. 6.37. FIXING END FRAMES

the end frame is kept parallel to its length, should be built in advance by atleast one course than the wall at whose end the end frme is fixed at right angles. This is essential so as to permit the string of brick-layers threads from the same end frame at perpendicular directions. In Fig. 6.37 (*d*), it may be seen that the corner does not impose any difficulty in fixing the end frame as the 10 mm dia. mild steel rod can be easily passed through mortar joint thickness.

(d) At T-junctions

In buildings there are longitudinal walls and cross-walls. Generally, for bonding the cross-walls, some tooths are left in the longitudinal wall which is built prior to it. For building the cross-wall, the end frames are fixed parallel to wall length at a distance of about 150 mm from junction as shown in Fig. 6.38. In this case, the cross-wall has been stopped at an opening and thus, the end-frame has been shown fitted at the end of the cross-wall abutting along its width. In case there is no opening and the cross-wall is solid, the end-frames near both ends could be fixed parallel to the length of the wall.

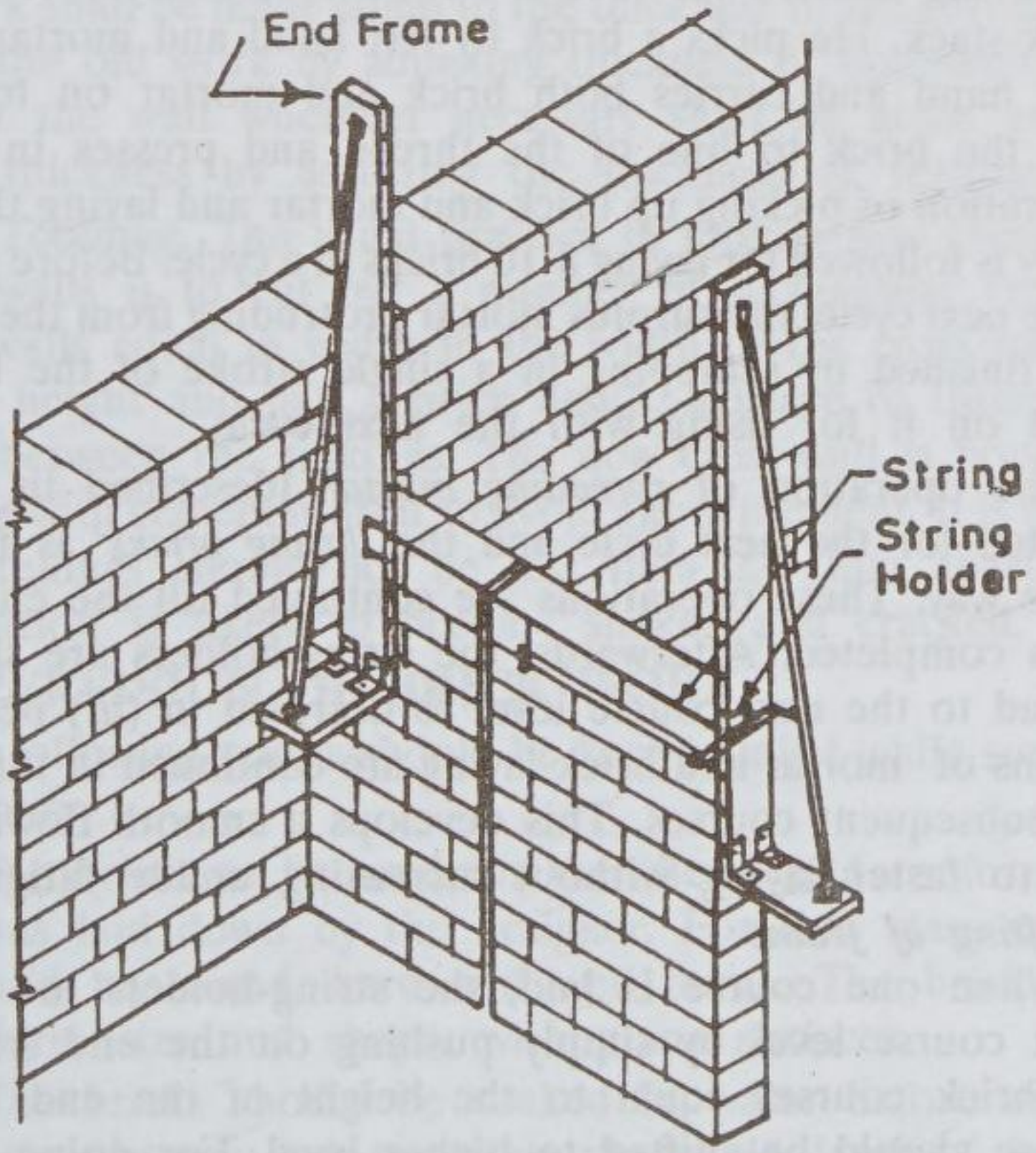


FIG. 6.38. FIXING AND USE OF END FRAMES FOR CROSS-WALL CONSTRUCTION.

4. Method of working

To break the joints in brick masonry, cut bricks (*i.e.*, closers) are required in alternate courses at the corners. It is therefore recom-

mended that the brick-layer should cut approximately the required number of bricks and arrange the same at the corner stacks of bricks, rather than cutting each time when needed. Afterwards, the end frames are fixed at corners and other openings as per requirement, as described above. The string holders are positioned at the appropriate course level and thread kept to line. The brick laying operations are carried out as described below.

(i) Spreading mortar

The brick layer picks up mortar on the trowel in right hand from the mortar board at one corner and unloads on the wall. The picking and unloading of mortar is carried at a stretch by the brick layer moving forward for a length of about a metre or so (to place 8-10 bricks) at a time. Then while moving backward, he spreads the mortar to level in a continuous stroke of the trowel. The unloading of mortar to longer length and the spreading stroke in one stretch allow the brick-layer to develop speed.

(ii) Laying bricks

Having levelled the mortar bed, the brick-layer turns towards the brick-stack. He picks a brick by left hand and mortar or trowel by right hand and carries both brick and mortar on to the wall. He lays the brick to line of the thread and presses in position. The operation of picking up brick and mortar and laying them simultaneously is followed for laying 8-10 bricks in a cycle. Before proceeding to lay the next cycle, the surplus mortar protruding from the horizontal joint is finished by scrapping in a single stroke of the trowel and collected on it for using with the next cycle.

The operation of *spreading mortar* [described in (*i*) above] is repeated for the next cycle and the *'laying bricks'* is followed in the same way. These operations are continued till the entire course length is completed. Afterwards, the string-holders are shifted with the thread to the next course level as desribed in (*iii*) below. These operations of mortar and brick-laying are continued in this sequence for the subsequent courses. This develops a smooth flowing rhythm leading to faster laying without increasing undue fatigue.

(iii) Shifting of frame

When one course is laid, the string-holders are shifted to the next course level by simply pushing on the end frame, when all the brick courses equal to the height of the end frames are laid, these should be shifted to higher level. For doing so, a joint at a distance of about 750 mm from the end frame is kept unfilled with mortar at a level of about 250 mm below the top of the end frame (marked with two lines), to position the mild steel flat for refixing the end frame. The end frames are removed from the existing position by loosening the butterfly nuts and removing the mild steel tie rods. The mild steel flat from the joint is taken out and placed

in the next position. The end-frame is checked for uprightness and alignment, and is secured to the wall with the help of mild steel tie rods and butterfly nuts. The string-holders are fixed in position in the usual way on the end frame.

5. Striking joints

(*i*) In cases where no pointing or plastering is required, the green mortar shall be neatly struck flush. Where pointing and plastering is required the joints should be racked out to a depth of not less than 10 mm.

(*ii*) Plaster work on the walls shall be deferred for a period preferably not less than 28 days sufficient to let shrinkage in reinforced concrete and masonry take place before plastering.

(*iii*) The face of brick work shall be cleaned and mortar dropping removed the very day that brick work is laid.

6. Joining old brick work with new brick work

(*i*) Joining shall be done in such a way that there shall not be any hump or projection at the joint. The thickness of each course of new work shall be made equal to the thickness of the corresponding course of the old work by adjusting thickness of horizontal mortar joints, and the wall wherever necessary shall be made exactly to the same thickness by adjusting the thickness of vertical joints.

(*ii*) *Toothing.* The usual practice in joining new cross-wall to old main walls is to cut out a number of rectangular recesses in the main walls equal in width to the width of the cross-wall, three courses in height and half a brick depth, a space of three courses being left between the sinkings. The new cross-wall is bonded into the recesses with cement mortar to avoid any settlement. It is necessary that the sinkings should not be less than 225 mm apart, as the cutting portion is likely to become shaken and cracked.

6.22. SUPERVISION OF BRICK WORK

The following points should be kept in mind while supervising brick masonry :

1. The bricks to be used for the site should conform to the specifications laid down by the designer. For first class work, the bricks should be sound, hard and well burnt. The bricks should be of uniform size and shape, with plane surfaces.

2. The bricks should be soaked in water before use for a period for the water to just penetrate the whole depth of the bricks. This period of soaking may be easily found at site by a field test in which bricks are soaked in water for different periods and then broken to find the extent of water penetration. The least period that corresponds to complete soaking will be the one to be allowed for in the construction work. When bricks are soaked, they should be removed from the tank sufficiently early so that at the time of

laying, they are skin dry. Such soaked bricks should be stacked on a clean place, where they are not spoilt by dirt, earth etc.

When mud mortar or fat lime mortar is to be used, bricks should not be soaked in water before use.

3. The bricks should be properly laid on their beds. They should be so laid that the frog is on the top surface. The mortar should cover completely the bed and the sides on the bricks. The bricks should be lightly pressed into the bed mortar so that uniform joint thickness is obtained.

4. The bricks, while laying, should be pushed sideways, to have uniform thickness of vertical joints. All joints should be properly flushed and filled with mortar of greater consistency so that no cavity is left in between.

5. For the thicker walls, the joints should be *grouted* in every course in addition to the bedding and flushing.

6. The brick work should be carried out perfectly in line. Ends or corners of the wall should be constructed first.

7. The brick work should be perfectly in level.

8. The brick work should be truly in plumb. The vertical faces should be checked by means of a plumb bob and the inclined surfaces, if any should be checked by means of wooden templates.

9. The brick work should be done in proper bond suggested by the designer.

10. Use of brick bats should be minimum. They should be used only where these are essential from bond point of view.

11. The mortar to be used should be of specified quality. Old mortar should not be used.

12. The brick work should be raised uniformly. The difference in heights, at any stage, between adjacent walls, should not be more than 1 m.

13. Where cross-wall is to be inserted later, steps or toothing or recesses should be provided during construction.

14. At plinth, window sill, floor or roof level and at the top of the parapet wall, the bricks course should be laid with bricks on edge.

15. When piers are tied up or buttersses, counterforts are used with wall, they should be built up course by course, so as to maintain proper bond with the main wall.

16. Iron fixtures such as hold fasts for doors etc., should be embeded in cement mortar or in cement concrete.

17. All the joints of the wall face (to be plastered later) should be raked to a minimum depth of 10 to 15 mm when the mortar is still green.

18. Where plastering or pointing is not to be done, the mortar joints should not be raked. They should be struck flush and finished at the time of laying.

19. After construction, the brick work should be kept wet for one to two weeks.

20. It is desirable to provide about 18 mm to 25 mm thick expansion joints after every 30 to 45 m length of the wall.

21. For carrying out brick work at higher level, single scaffolding should be adopted. This is done by removing required headers from the wall to provide supports for the scaffolding. The removed headers are repacked later when scaffolding is removed.

6.23. COMPARISON OF BRICK MASONRY AND STONE MASONRY

(a) Points in favour of brick masonry

1. Brick work is cheaper at places where stones are not available. If stones are available at some distance, the transportation costs are very high.

2. Generally, brick masonry can be constructed with less skilled masons, in comparison to stone work. Hence brick work is cheaper.

3. Bricks are easy to handle. They can be lifted by manual labour. No special lifting arrangement is required.

4. Brick masonry can be constructed in any type of mortar. For low rise houses, where the loads are moderate, even mud mortar can be used which is cheaply available.

5. Bricks are of regular size and shape. Due to this proper bond can be maintained. Stones require dressing for maintaining the bond.

6. Brick work requires lesser mortar because of thin mortar joints required.

7. Because of plane surface obtained, the thickness of plaster in brick work is much less than in stone work.

8. Since bricks are in regular sizes, thinner walls can be constructed. In bricks, single brick thick walls (20 cm) can be constructed while in stone masonry, it is difficult to construct walls of thickness lesser than 30 cm.

9. The dead load of the walls is much less in brick masonry than in the stone masonry, because of lesser minimum thickness of walls. This is important factor in the area, where the bearing capacity of soils is low.

10. It is easy to form openings to construct connections in brick work. In stone work, dressing of stones is required to achieve this.

11. Bricks are better fire-resistant than stones. Bricks do not easily disintegrate.

12. Good quality bricks can resist the various atmospheric effects in much better way than stones. Brick walls are relatively cooler than the stone walls.

(*b*) Points in favour of stone masonry

1. Stone masonry is stronger than bricks masonry of the same wall thickness. Their load-carring capacity is more.

2. The life of stone masonry is much more than the bricks masonry.

3. Stone work gives massive appearance. Due to this, public buildings and monumental works are preferred in stone masonry.

4. Stone masonry does not require external plaster. Due to this, the maintenance cost is less.

5. Better architectural effects can be given in stone work.

6. Stone masonry, per unit volume, is cheaper, where stones is readily available.

7. There is no requirement for fuels etc. for stones, as required for the preparation of bricks. If these fuels are in short supply, the work may come to stand still.

8. Stone work is more water tight than brick masonry. Bricks absorb moisture from atmosphere, due to which dampness can enter the buildings and even damage the internal finishes.

6.24. DEFECTS IN BRICK MASONRY

Brick masonry may develop the defects due to the following reasons :

(*i*) Sulphate attack
(*ii*) Crystallization of salts from bricks (efflorescence)
(*iii*) Corrosion of embedded fixtures
(*iv*) Drying shrinkage.

1. *Sulphate attack*

This is a common defect, specially at locations where the brick work is either exposed (such as in boundary walls, unplastered external walls etc.) or, where brick work is likely to come in contact with moisture. The sulphate salts present in brick react with hydraulic lime in the case of lime mortar and with alumina of cement in the case of cement mortar. Due to this reaction, the increase in the volume of mortar takes place, resulting in chipping and spalling of bricks. Cracks are formed in joints and rendering.

2. *Crystallization of salts from bricks*

If the bricks are manufactured from earth containing excessive soluble salts, entry of moisture, either due to dampness or due to rains etc, dissolves the soluble salts. These salts, after getting dissolved in water, appear in the form of fine whitish crystals on the exposed

brick surface. This is known as efflorescence. Such a masonry presents ugly appearance. The situation can be improved by brushing and washing the affected surface from time to time.

3. Corrosion of embedded fixtures

Iron or steel fixtures, such as the pipes or holdfasts of doors, windows etc., embeded in brick masonry gets corroded with time specially when lime mortar is used. The corrosion results in the increase in the volume, resulting in cracks in brick masonry. Therefore, these fixtures should be well-embedded in cement mortar.

4. Drying shrinkage

When moisture penetrates the brick work, it swells. On evaporation of moisture during the drying due to atmospheric heat etc., the bricks shrinks, resulting in the development of cracks in the masonry joints. Frequent swelling and shrinkage may cause even the fatigue of masonry.

6.25. STRENGTH OF BRICK MASONRY

The strength of brick masonry depends upon the following factors :

1. Type and quality of bricks.
2. Mortar mix proportion.
3. Size and shape of masonry construction.

Type and quality of bricks

The strength of brick masonry primarily depends upon the type and class of bricks used, and the basic compressive strength of bricks. Strength of bricks in India varies from region to region depending upon the nature of available soil used for bricks and technique adopted for moulding and burning. Some research has been done for manufacture of bricks of improved quality from soils, such as black cotton moorum, which ordinarily gives bricks of very low strength. Table 6.1, based on information collected by BIS

TABLE 6.1

	Area	*Compressive strength of bricks*	
		(kg/cm^2)	(N/mm^2)
1.	Delhi and punjab	70 to 100	7 to 10
2.	Uttar Pradesh	100 to 200	10 to 20
3.	Madhya Pradesh	35 to 50	3.5 to 5
4.	Maharashtra	50	5
5.	Gujarat	30 to 100	3 to 10
6.	Rajasthan	30	3
7.	West Bengal	100 to 200	10 to 20
8.	Andhra Pradesh	30	3
9.	Assam	35	3.5

gives the general idea of the average strength available in various parts of India, employing commonly known methods for moulding and burning. In certain cities like Delhi, Calcutta and Madras, machine-made bricks are now being produced, which give compressive strength varying between 175 to 200 kg/cm^2 (17.5 to 20 N/mm^2).

Before designing the brick masonry structures, it is essential to determine the compressive strength of brick units.

Following relation generally holds good between strength of bricks and number of storeys in case of simple residential buildings having one brick thick walls (20 cm) and rooms of moderate size:

TABLE 6.2

Compressive strength		*Storeys*
(kg/cm^2)	(N/mm^2)	
30–35	3–3.5	1 to 2
70	7	2 to 3
105	10.5	3 to 4
140	14	4 to 5

Mortar mix proportion

Type of mortar and mix proportion is another important factor which determines the strength of masonry. The strength of various types of mortars has been discussed in chapter 5. Table 5.1 gives the compressive strength of masonry lime mortar of various mix proportions. Table 5.2 gives the compressive strength of cement mortars, while Table 5.3 gives compressive strength of gauged mortars.

Size and shape of masonry construction

The strength of brick masonry walls depends upon (*i*) slenderness ratio of masonry, and (*ii*) shape factor. For a wall, the *slenderness ratio* is taken as the effective height of the wall divided by its effective thickness, or the effective length divided by the effective thickness, whichever is less. For a column, the slenderness ratio is equal to the effective height divided by the corresponding lateral dimension (thickness or width). These terms have been more elaborately defined and discussed in chapter 8. *Shape factor* takes into account the effect of shape of the brick, *i.e.* ratio of its height to thickness.

Table 6.3 gives the *stress factors* for various slenderness ratio. The values of basic compressive strength of brick masonry given in Table 6.5 should be multiplied by these stress factors.

TABLE 6.3
STRESS FACTOR FOR SLENDERNESS RATIO

S.No.	*Slenderness ratio*	*Stress factor*
1	6	1.000
2.	8	0.920
3.	10	0.835
4.	12	0.750
5.	14	0.660
6.	16	0.565
7.	18	0.480
8.	21	0.448
9.	24	0.415

TABLE 6.4
MODIFICATION FACTOR FOR SHAPE OF BRICK

Ratio of height to thickness of brick or block	*Factor*
0.75	1.0
1.0	1.2
1.5	1.6
2.0 to 3.0	2.0

The values of basic stresses (Table 6.5) are suitable when the units are of common brick shape, but may be unnecessarily low for same units whose ratio of height to thickness is greater than that of common brick. For units of crushing strength not greater than 55 kg/cm^2 (5.5 N/mm^2) and with a ratio of height to thickness as laid greater than 0.75 but not greater than 3, the *basic stress* (Table 6.5) may be modified by the factors specified in Table 6.4.

Permissible compressive stress of brick masonry

Table 6.5 gives the safe or permissible compressive stress for brick masonry using bricks of various basic stress and for various types of mortars. The permissible compressive stresses recommended in the table apply to masonry walls consisting of squared units built to horizontal courses, with broken vertical joints. The effects of slenderness ratio and shape factor should be taken into consideration as explained above.

The following *notes* refer to table 6.5.

Note 1. The table is valid for slenderness ratio 6 and loading with zero eccentricity.

Note 2. Linear interpolation is permissible for units whose

TABLE 6.5 BASIC COMPRESSIVE STRESSES FOR MASONRY MEMBERS

S.N.	*Description of mortar*	*Mix (Parts by volume)*					*Hardening time after completion of work*	*Basic compressive stress in kg/cm² corresponding to masonry units having crushing strength (kg/ cm²)*					
		Cement	*Lime*	*Lime-Pozzolana*	*Pozzolana*	*Sand*		*35*	*70*	*105*	*140*	*175*	*210*
1.	Cement	1	0 – $\frac{1}{4}$ (C)	—	—	3	7	3.5	7.0	10.5	12.5	14.5	16.5
2.	Cement	1	$\frac{1}{2}$(C)	—	—	$4\frac{1}{2}$	14	3.5	7.0	10.0	11.5	13.0	14 5
3.	Cement-lime	1	1(C)	—	—	6	14	3.5	7.0	10.0	11.0	12.0	13.0
4.	Cement-lime	1	2(B)	—	—	9	14	3.5	5.5	8.5	10.0	11.0	12.0
5.	Cement	1	—	—	—	6							
6.	Lime-Pozzolana mixtured	—	—	1	—	1.5							
7.	Cement-lime	1	3(B) or (C)	—	—	12	14	2.5	5.0	7.0	8.0	9.0	10.0
8.	Hydraulic lime	—	1(A)	—	—	2	14	2.5	5.0	7.0	8.0	9.0	10.0
9.	Lime-pozzolana	—	1(C)	—	1	2							
10.	Lime	—	1(B)	—	—	3	28	2.5	4.0	5.5	6.0	6.5	7.0

crushing strengths are intermediate between those given in the Table.

Note 3. Lime classification (classes A, B and C) and building lime shall conform to accepted standards.

Note 4. For mortar under serial No. 6, lime pozzolana mixture shall be of grade *LP* 40 conforming to accepted standards.

Tensile stress in masonry

In general no reliance shall be placed on the tensile strength of brick work in the calculations. The designer should assume that, that part of the section will be inactive and the remainder will carry compressive stress only. However, for mortars not weaker than 1 : 1 : 6 : : cement : Lime : sand mix or its equivalent, the permissible tensile stress in bending shall not exceed 1 kg/cm^2 (0.1 N/mm^2).

Permissible shear stress

In the case of walls built in mortar not weaker than 1 : 1 : 6 : : cement : lime : sand mix and resisting horizontal forces in the plane of the wall, the permissible shear stress, calculated on the area of horizontal mortar bed joint, shall be taken as 1.5 kg/cm^2 (0.15 N/mm^2 or 150 kN/m^2).

Brick masonry in mud mortar

Table 6.5 does not include the permissible compressive strength of brick masonry in mud mortar. For such a work, the safe compressive strength may be taken as 1.5 kg/cm^2 (15 t/m^2 or 150 kN/m^2)

6.26. THICKNESS OF A BRICK WALL

The thickness of a brick wall depends upon the following:

1. Super-imposed load per unit length of the wall,
2. Overall height of the wall,
3. Height of the wall between floors.
4. Length of the wall between piers, buttresses, crosswalls (*i.e.*, lateral support conditions), and
5. Strength of brick masonry, which depends upon the quality of bricks, quality of mortar and method of bonding.

The *structural desisgn* of masonry wall is done by 'calculated masonry method'. The method has been explained in chapter 8.

In absence of any detailed calculations, the minimum thickness recommended in Table 6.6 may be adopted. The following points should be kept in mind while using Table 6.6 :

1. Height of each storey is not more than 4.8 m

2. The *length of wall* is the length measured between buttresses or cross-walls, which are properly bonded to the main wall, so that sufficient lateral support is available.

TABLE 6.6
MINIMUM THICHNESS OF EXTERNAL AND PARTY MASONRY WALLS OF RESIDENTIAL AND BUSINESS BUILDINGS

Storey above ground	*Height of wall in metres above plinth level*		*Length of wall (m)*	*Thickness of wall (cm)*							
	Exceeding (m)	*Not exceeding (m)*		*Basement Floor*	*Ground Floor*	*First Floor*	*Second Floor*	*Third Floor*	*Fourth Floor*	*Fifth Floor*	*Sixth Floor*
1	–	3.5	Any	30	20						
1	3.5	5.0	Any	40	30						
1	5.0	6.5	Any	50	40						
2	–	6.5	Under 10	30	20	20					
2	–	6.5	Over 10	40	30	20					
2	6.5	9.5	Under 10	40	30	30					
2	6.5	9.5	Over 10	50	40	30					
3	–	10.0	Under 10	40	30	20	20				
3	–	10.0	Over 10	50	40	30	20				
3	10.0	13.5	Under 10	50	40	30	30				
3	10.0	13.5	Over 10	60	50	40	30				
4	–	13.5	Under 10	50	40	30	30	20			
4	–	13.5	Over 10	60	50	40	30	30			
4	13.5	18.0	Under 10	60	50	40	30	30			
4	13.5	18.0	Over 10	70	60	50	40	30			
5	–	16.5	Under 10	60	50	40	30	30	30		
5	–	16.5	Over 10	70	60	50	40	30	30		
5	16.5	23.0	Under 10	70	60	50	40	30	30		
5	16.5	23.0	Over 10	80	70	60	50	40	30		
6	–	20.0	Under 10	70	60	50	40	30	30	30	
6	–	20.0	Over 10	80	70	60	50	40	30	30	
6	20.0	27.5	Under 10	80	70	60	50	40	40	30	
6	20.0	27.5	Over 10	90	80	70	60	50	40	30	
7	–	23.5	Under 10	80	70	60	50	40	30	30	30
7	–	23.5	Over 10	90	80	70	60	50	40	30	50
7	23.5	32.0	Under 10	90	80	70	60	50	40	40	30
7	23.5	32.0	Over 10	100	90	80	70	60	50	40	30

3. The thickness of wall should not be less than 1/6 of the storey height.

4. For basement walls, the thickness should not be less than one-third the height of retained soil above basement level, nor should it be less than the thickness of wall at ground floor plus 10 cm.

5. Table 6.6 is applicable for walls built of bricks or concrete blocks, using lime mortar (1 : 3), or cement mortar (1 : 6) or composite mortar (1 : 2 : 9).

6.27. TYPICAL STRUCTURES IN BRICK WORK

Following are the common structures constructed in brick-work:

1. Walls
2. Piers
3. Footings
4. Buttresses
5. Thresholds
6. Window sills
7. Corbels
8. Copings
9. Jambs
10. Ornamental brick work
11. Brick work curved in plan
12. Brick knogging
13. Retaining walls and breast walls
14. Fire places and flues
15. Chimneys
16. Arches
17. Lintels
18. Cavity walls.

Out of these, walls, piers and footings have already been discussed in earlier articles of this chapter. Fire places and flues, chimneys, arches, lintels and cavity walls have been discussed in separate chapters.

6.28. BUTRESSES

Buttresses are piers that are provided to resist thrusts from roof trusses or strengthen main walls or boundary walls. They give lateral support to the main load bearing walls. They are usually in the form of projections and are usually completed with cappings. Two forms of cappings : (*i*) splayed capping, and (*ii*) tumbled-in-capping are shown in Fig. 6.39.

Buttresses are usually designed to resist overturning moment due to lateral thrust. Their thickness is found in such a way that the resultant of the vertical and lateral loads remain within the middle third of the section so that no tension is developed. Buttresses

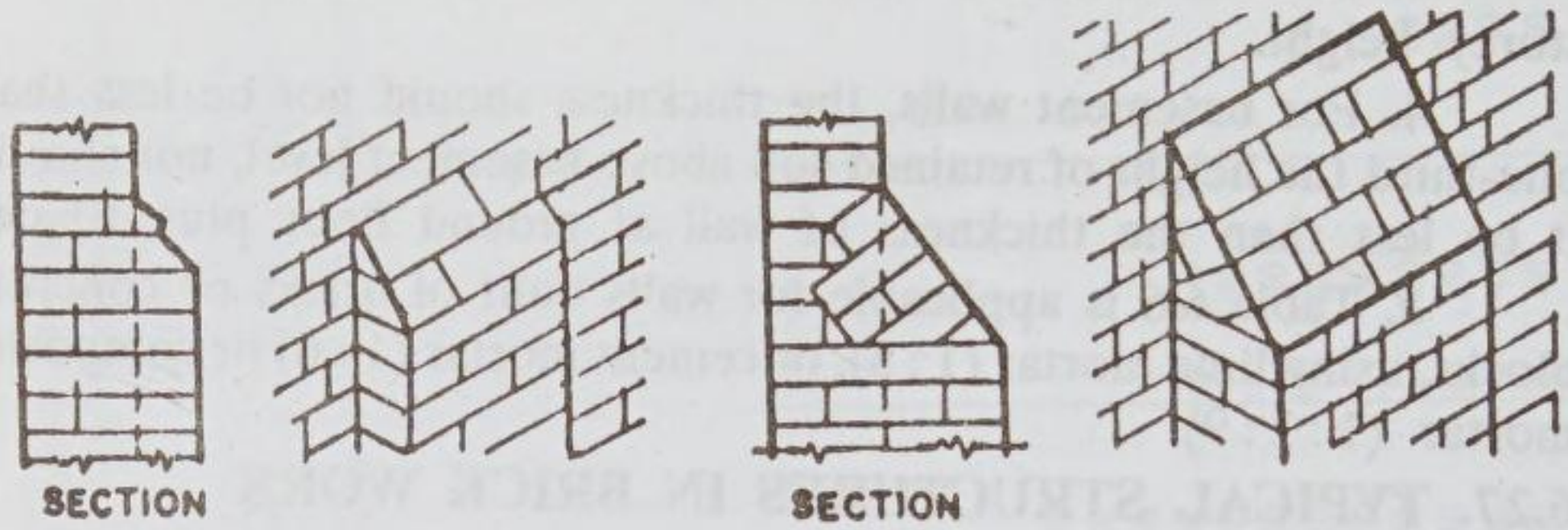


FIG. 6.39. BUTTRESSES

must be constructed along with the walls so that they are bonded to the wall course by course.

6.29. THRESHOLDS

Threshold consists of the arrangement of one or more steps outside the external door opening. Two forms of thresholds are shown in Fig. 6.40. Each step of the threshold should be constructed with slight outward-slope so that the rain water can be easily drained

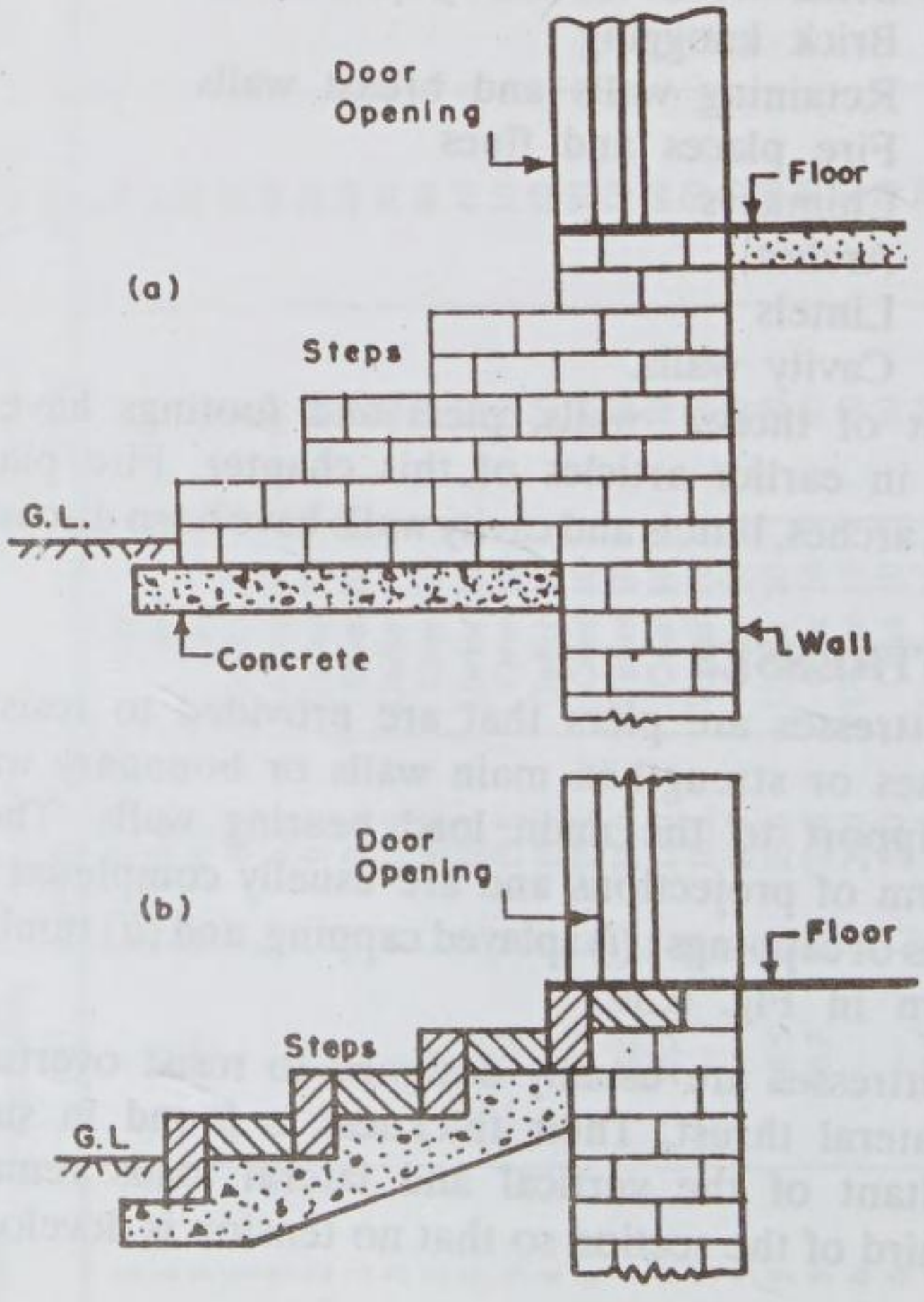


FIG. 6.40. THRESHOLDS.

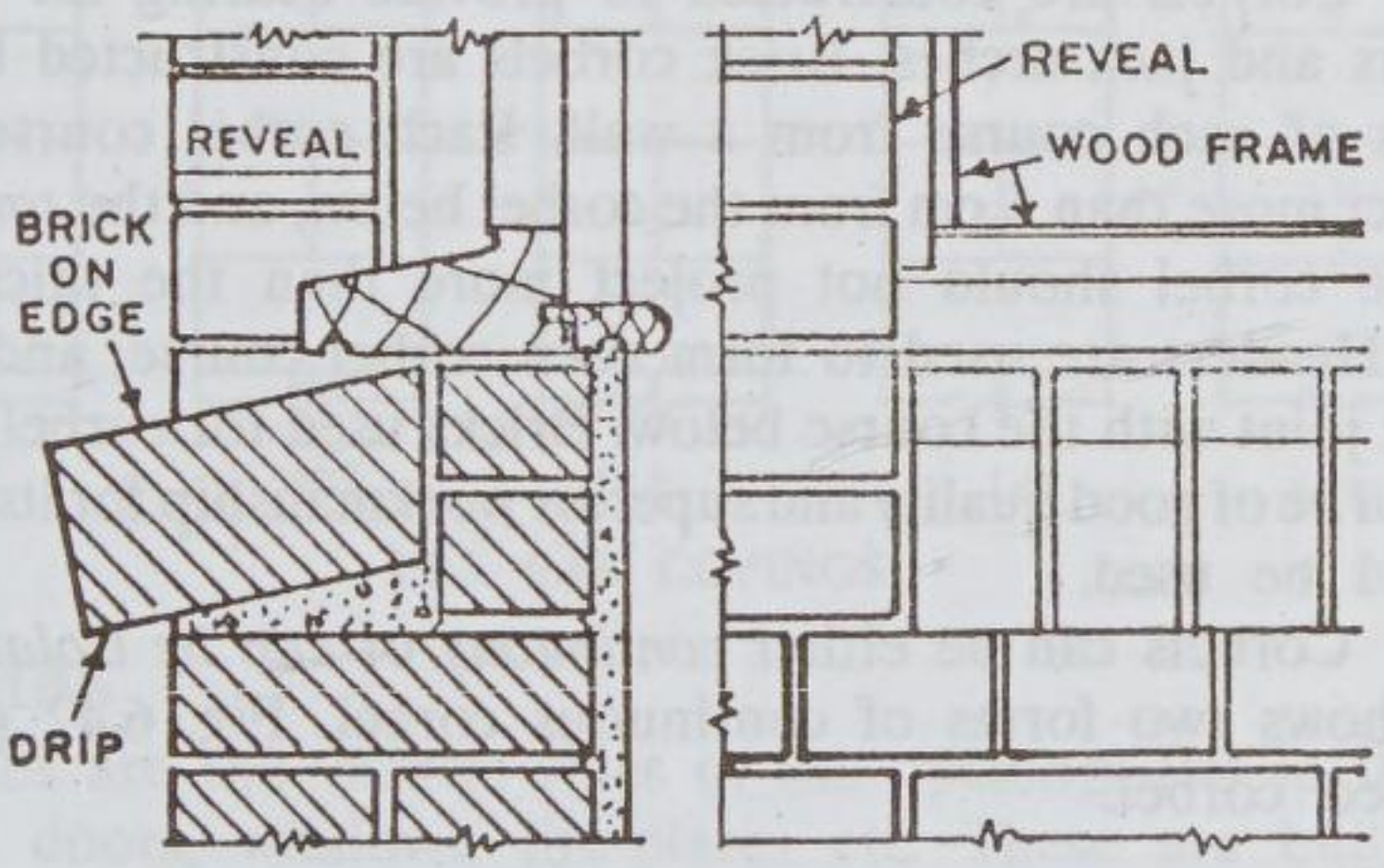
off. The construction should be done in cement mortar. It is preferable to use some sort of hard finishing on the top of each step. Thresholds are constructed at the last stage of building construction, when other construction activities have almost come to an end.

6.30. WINDOW SILLS

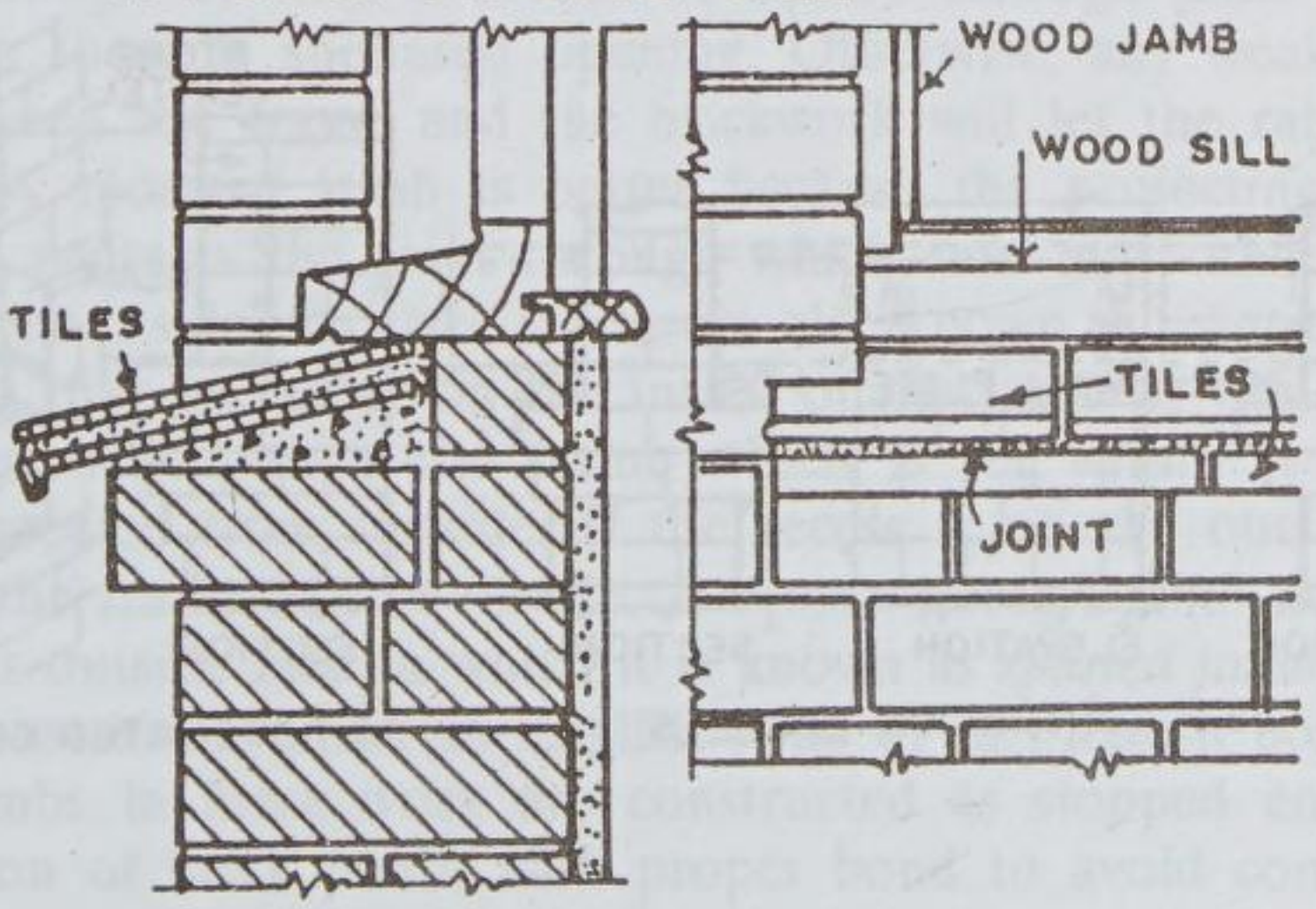
A sill provides a suitable finish to the window opening and it affords a protection to the wall below. A great many external sills in modern buildings are constructed of bricks laid on edge, or of roofing tiles, both of which harmonize well with brick walling. Fig. 6.41 shows vertical section and part elevation of two type of sills.

The following points should be kept in mind in constructing brick sills :

1. The sills of windows, on external walls, should be properly weathered (slope 1 in 6) to drain off rain water. The projection



(a) BRICK ON EDGE SILL



(b) TILE SILL

FIG. 6.41. BRICK AND TILE SILLS.

of sill, if any, should not be less than 50 mm and should be suitably throated.

2. Bricks for the sills should be hard, well burnt and set in cement motar.

3. The top surface of the brick sills should be provided with suitable finish.

4. In sills made of tiles, tiles are laid in cement mortar and in two courses, breaking joint as indicated in elevation (Fig. 6.41 *b*). The lower course of tiles should be provided with continuous nibs which form a perfect drip, past which no dripping rain water can find its way.

5. It is preferable to provide damp proofing course below the window sill so that moisture does not enter inside the structure.

6.31. CORBELS

Corbels are constructed to provide bearing for floor beams, girders and jack arches. Brick corbels are constructed by projecting bricks of each course from a wall. Each corbel course should not project more than 5 cm from the corbel below, and the *total* projection of the corbel should not project more than the thickness of the wall. Headers are used to form each corbel course, and they should break joint with the course below. Bricks used for corbel construction should be of good quality and superior workmanship for its construction should be used.

Corbels can be either *continuous* or *can be isolated*. Fig. 6.42 (*a*) shows two forms of continuous corbel. Fig. 6.42 (*b*) shows an isolated corbel.

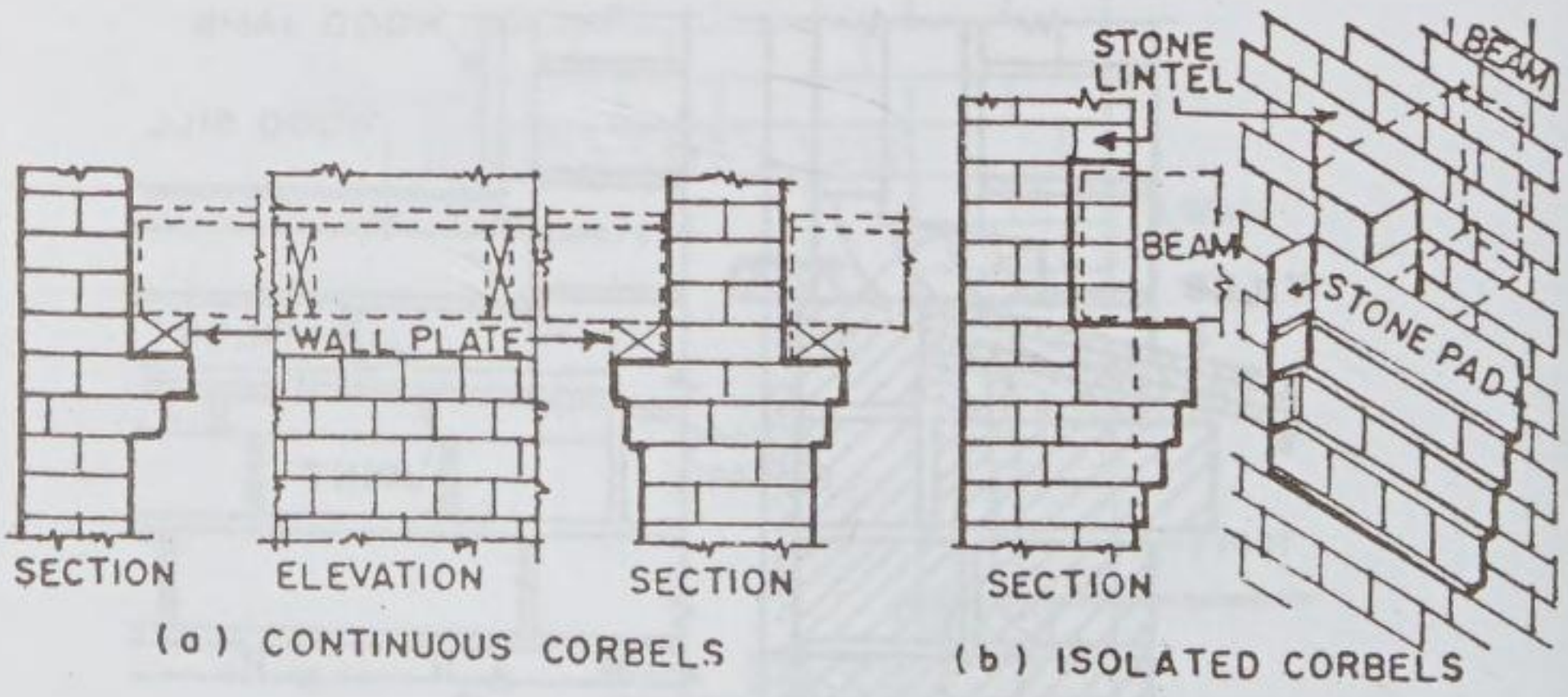


FIG. 6.42. CORBELS.

6.32. COPINGS

Copings are provided to serve as a protective coverings to walls at its top. Coping throws the rain water clear off the wall. Sometimes, special moulded bricks are used for coping, having proper weathering and throating. If copings are made of regular bricks, they are to be properly shaped. Bricks used for coping should be hard and strong enough to resist weathering actions. The joints in the coping should be fewer. They should be invariably constructed to cement mortar.

Fig. 6.43. shows some common types of brick copings.

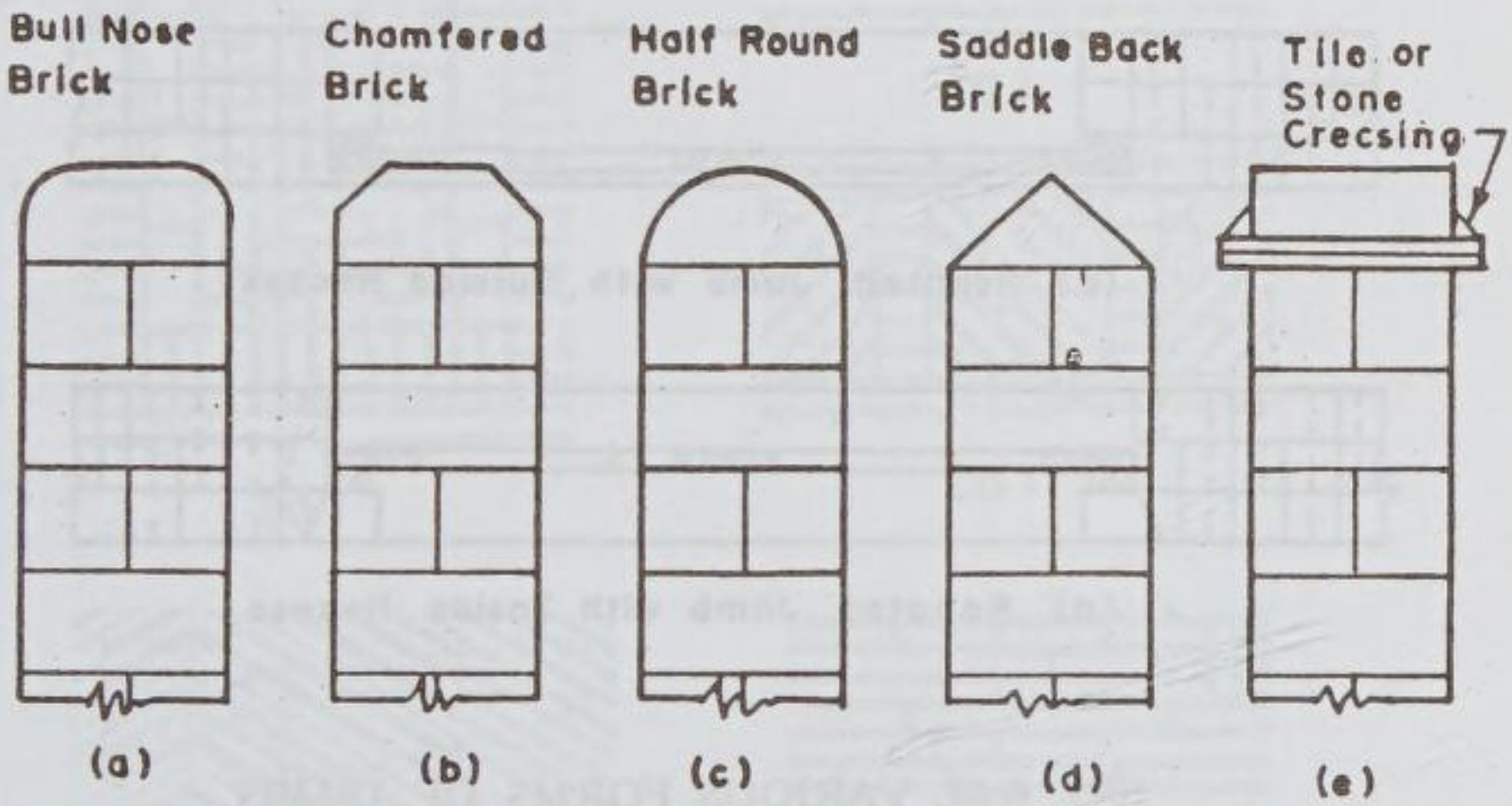
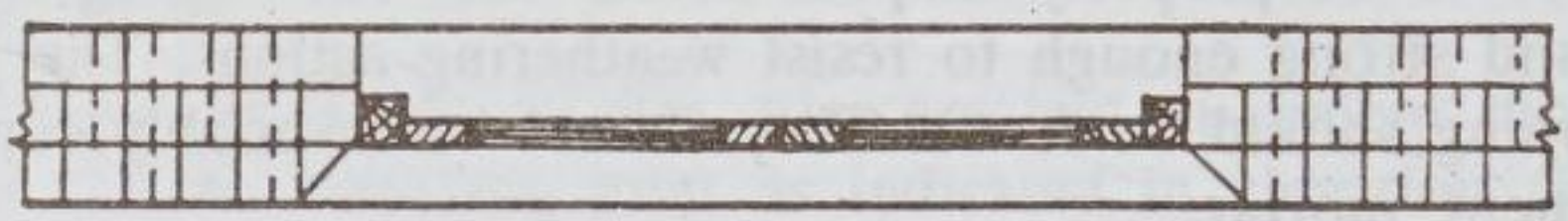


FIG. 6.43. COPINGS.

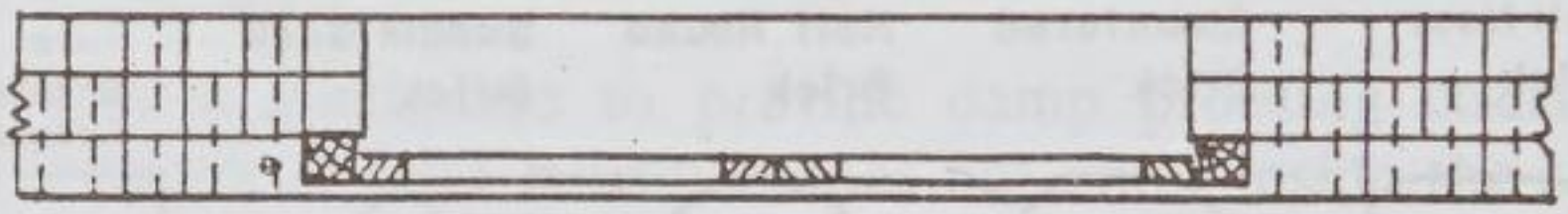
6.33. JAMBS

Jambs are the vertical sides of the openings left in the walls to receive doors, windows, fire-places etc. These are built either square through or with a recess. A square through jamb is used only when there is sheltered opening. Otherwise, any weakness in joint between the frame and the brickwork will let the rain water through. A recessed jamb is better because the projecting nib of brickwork protects the joint through which rain may otherwise be driven to the inside. *Recessed jambs* are also known as *rebated jumbs.* The recess may be either on the inside of the jamb or the outside. If it is on inside, then the frame which is set within it will be partly concealed from outside. If the recess is on the outside, the whole of the frame will be visible. A *square through jamb* may have splay at its outside face in which it is known as *splayed jamb.* Jambs may be constructed either in English bond or in Flemish bond. The square jambs in brick work are constructed as stopped ends. For construction of brick jambs with proper bond to avoid continuous vertical joints, it is essential to use bevelled bats and king, queen or bevelled closers.

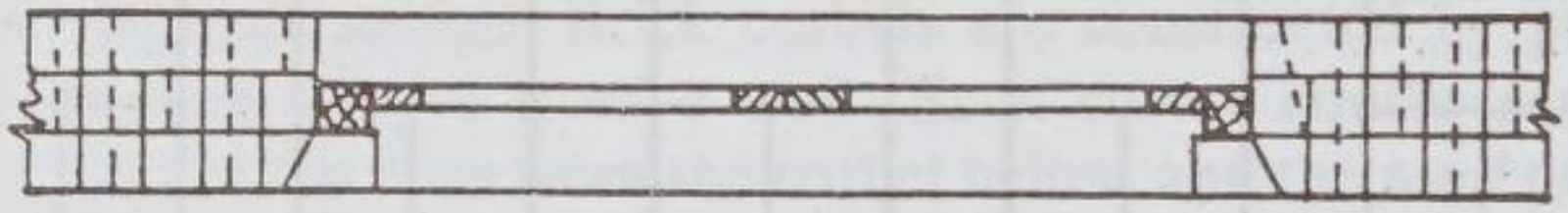
(a) Square – Through Jamb



(b) Splayed Jamb



(c) Rebated Jamb with Outside Recess



(d) Rebated Jamb with Inside Recess

FIG. 6.44. VARIOUS FORMS OF JAMBS.

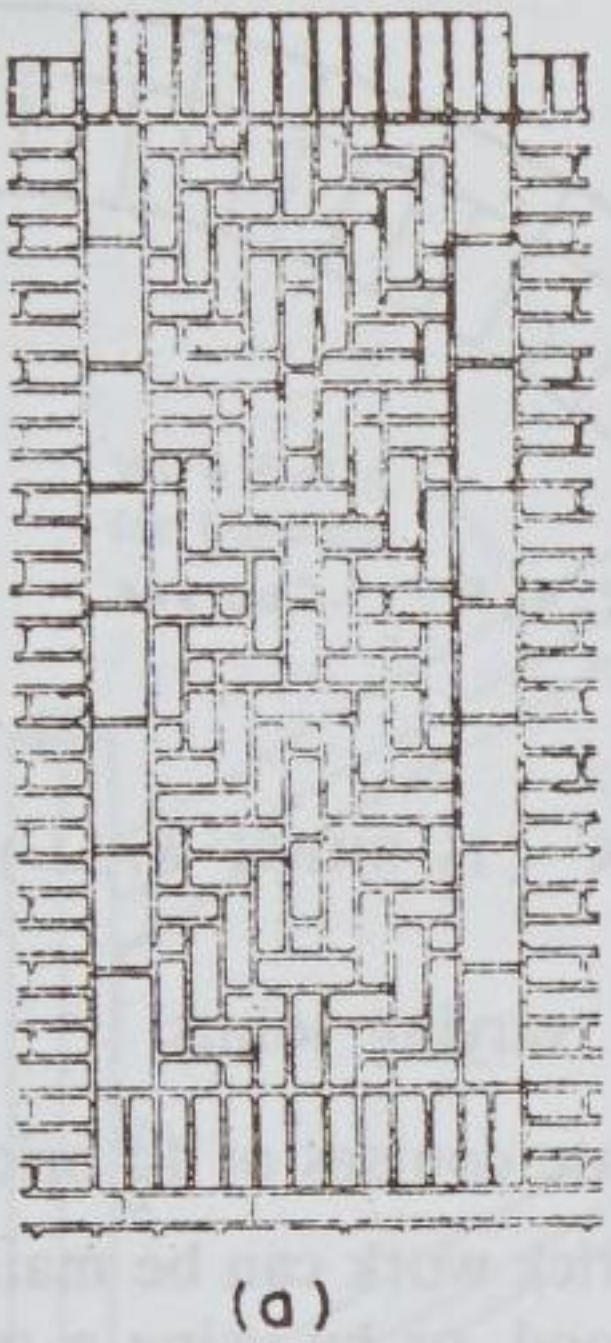
6.34. ORNAMENTAL BRICK WORK

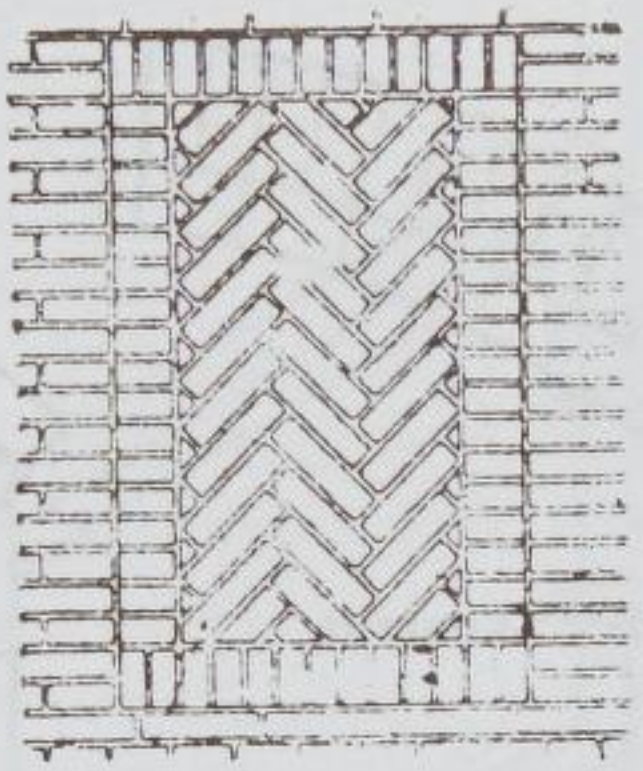
Ornamental brick work can be obtained by the use of special types of bricks (moulded bricks), mortars of different colours, mortar joints of different thickness and different arrangement of bricks, so as to get pleasing appearance. Sometimes, bricks of different thicknesses are used to give architectural treatment. Machine made bricks with sharp and angular faces present more pleasing appearance. Even coloured bricks can be used in a suitable pattern. Texture of the bricks is also important. Though sandy textured bricks give better appearance, but smooth face bricks are preferred in areas, where dust storms are more frequent. Recessed joints produce deep shadows and thus give better appearance. The ornamental brick work is used only for facia work. Sometimes, a combination of bricks, tiles and stones produce a much better effect.

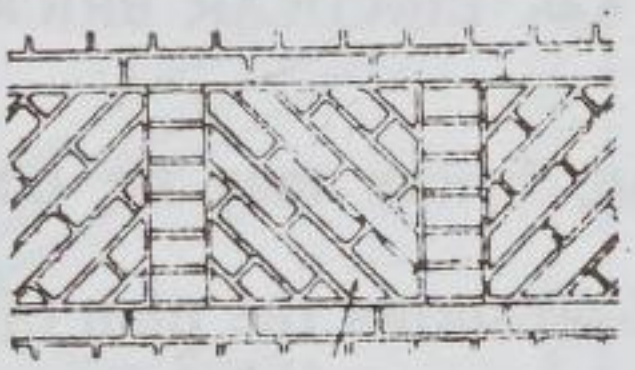
Fig. 6.45 gives few examples of ornamental brick work.

6.35. BRICK WORK CURVED IN PLAN

Brick work curved in plan is sometimes required, such as in construction of chimneys, soak pits, flues etc. Brick work curved







(a) VERTICAL PANELS (b) DIAGONAL PANEL (c) QUOINS

FIG. 6.45. EXAMPLES OF ORNAMENTAL BRICK WORK.

in plan is built exactly in the same manner as for general brick work, but where the inner radius is 6 metres or less, all courses should be of headers with bricks cut to radius. For large work, specially moulded bricks should be used in lieu of bricks cut to radius. Standard bricks, if used would give very wide joints. In case of unimportant works such as lining to soak pits and cess pools, circular brick works of inner radius less than 6 m may be built like brick work straight on plan or to a curve exceeding 6 metre inner radius. These specifications also apply to brick work polygonal in plan. Where water tightness is required, moulded bricks, or bricks cut to radius should be used. Where water tightness is not a major

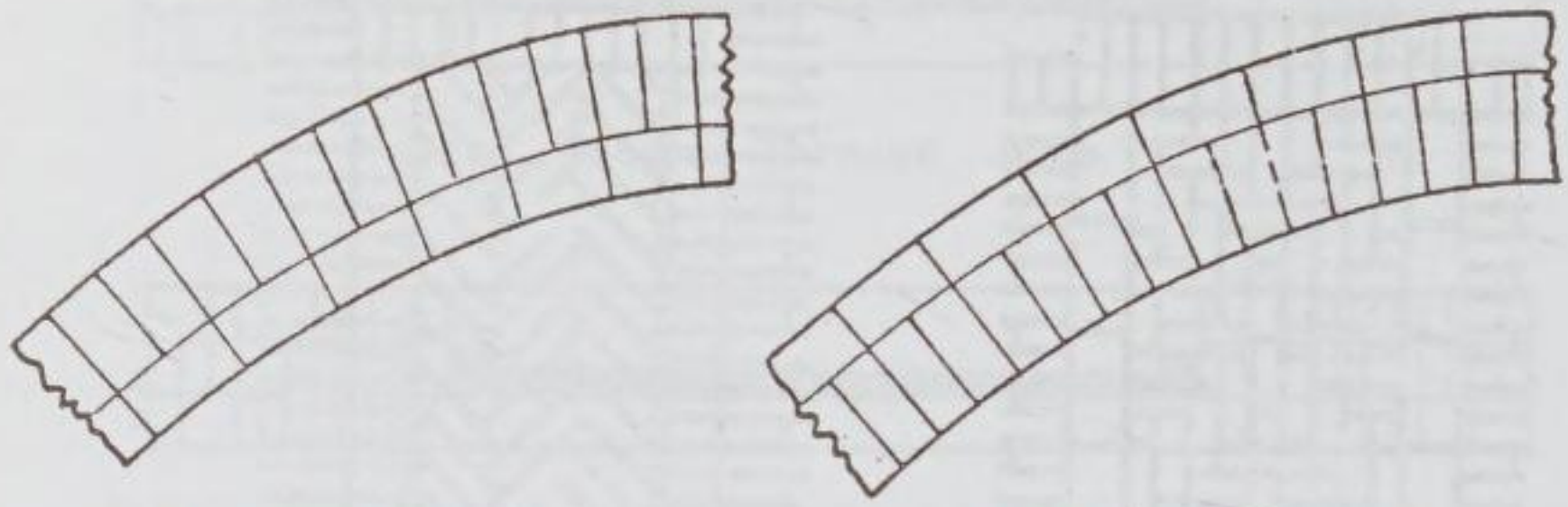


FIG. 6.46. CIRCULAR BRICK WORK ($1\frac{1}{2}$-BRICK WALL)

consideration, bricks may be laid with varying joints.

Fig. 6.40 shows the plan of alternate courses of $1\frac{1}{2}$ brick thick wall circular in plan. The shape of the brick work can be maintained either by a *template* of thin board of wood, or by using a *trammel*.

6.36. BRICK KNOGGING

Brick knogging is the term used to denote brick work built up between wooden quarters or framing. Fig. 6.47 shows *brick knogging*. The uprights or posts are 150 mm×120 mm in size, placed at a central distance of 1.50 metres apart. The horizontal members are *ribs* of planking (known as *knogging member*) 100 mm×50 mm, fixed at 900 mm vertical distance apart. All the faces of the timber in contact with the masonry is well-coated with boiling coaltar (two coats) and the faces of timber exposed to view, on completion, is given three coats of specified paint. The bricks are laid in the openings of the framework and are placed in such a way that equal projections of timber are left on both the sides. Brick work is done in lime or cement mortar. After the completion of brick work, the surfaces of brick work is kept thoroughly wetted before plastering. Nails are deiven into the ledge of the timber frame work to give a hold to the cement/lime plaster with which both faces of the brick work is then finished off, of a thickness to be flush with the faces of the posts. The plastering is cured for three weeks. If the wooden members are of shorter width and the entire exposed surface is to be plastered, a *metal lath* is fixed on both the sides of knogging and the entire area is then plastered. This arrangement will check the plaster from peeling off from the wooden members.

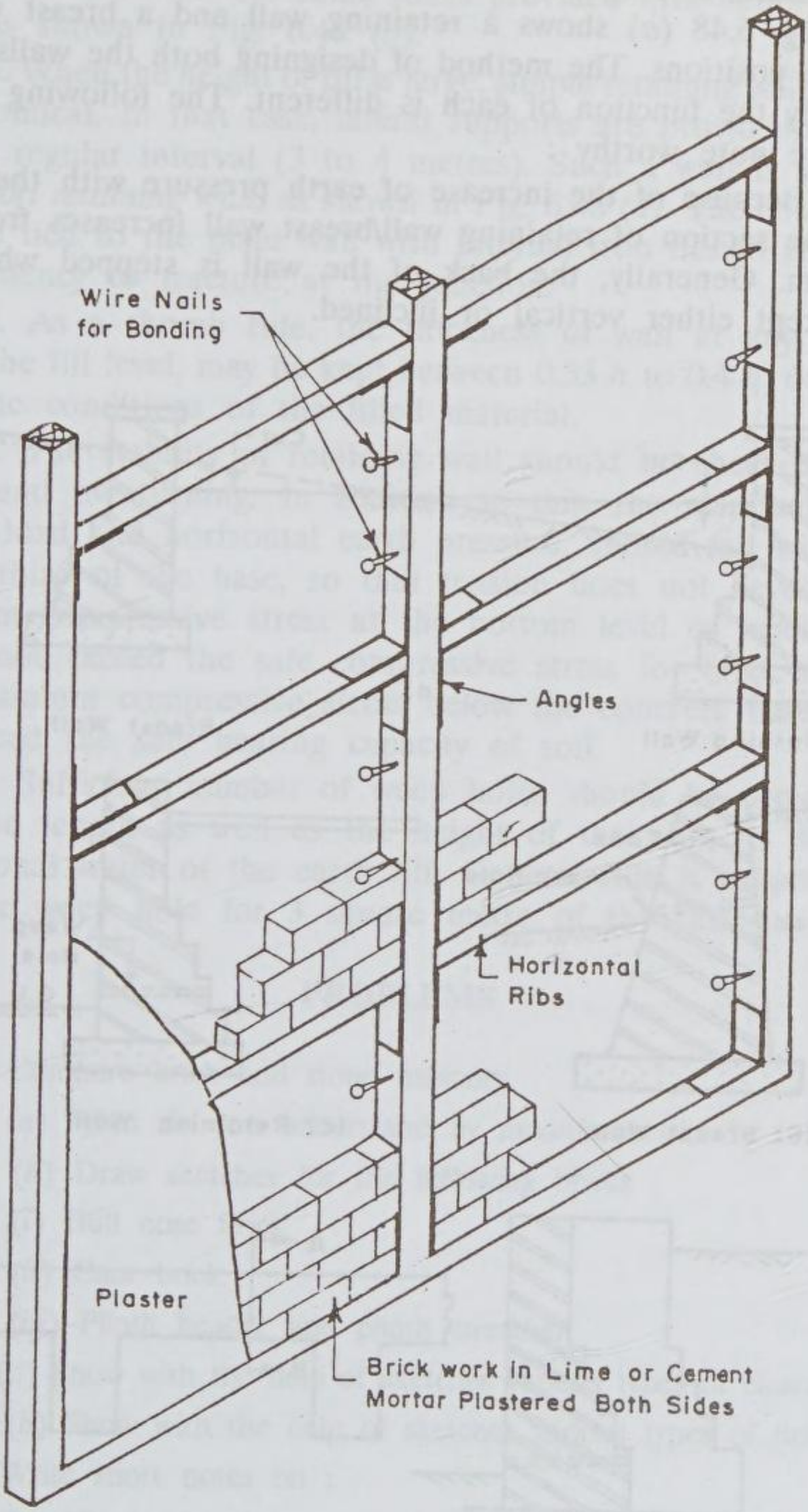


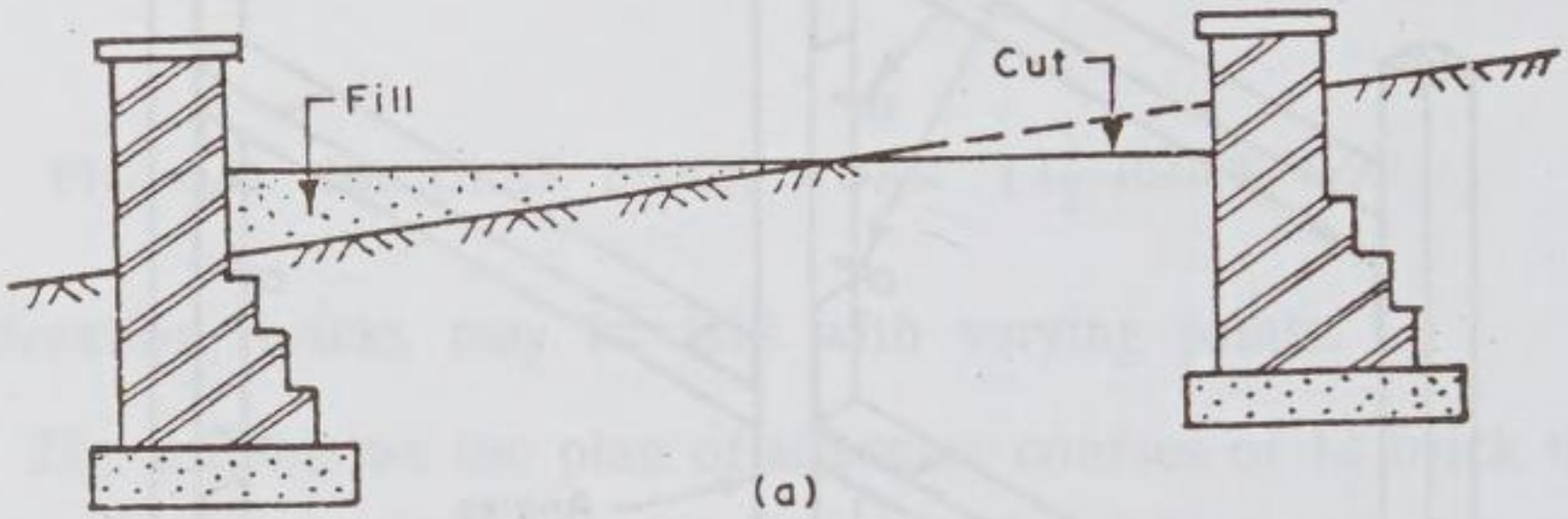
FIG. 6.47. BRICK KNOGGING.

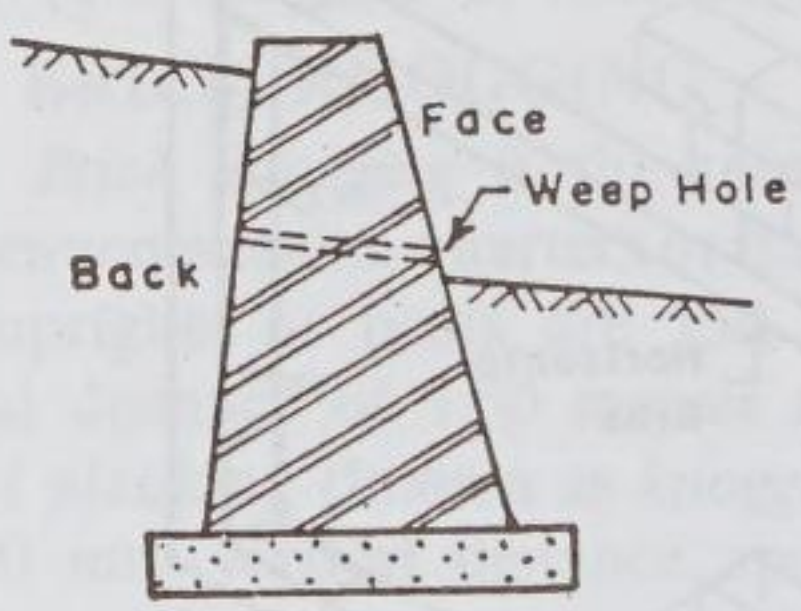
6.37. RETAINING WALLS AND BREAST WALLS

A *retaining wall* is a wall of increasing thickness, which is constructed to retain artificial filling (mostly earth fill) to one side. A *breast wall* is similar to retaining wall, but it is constructed to

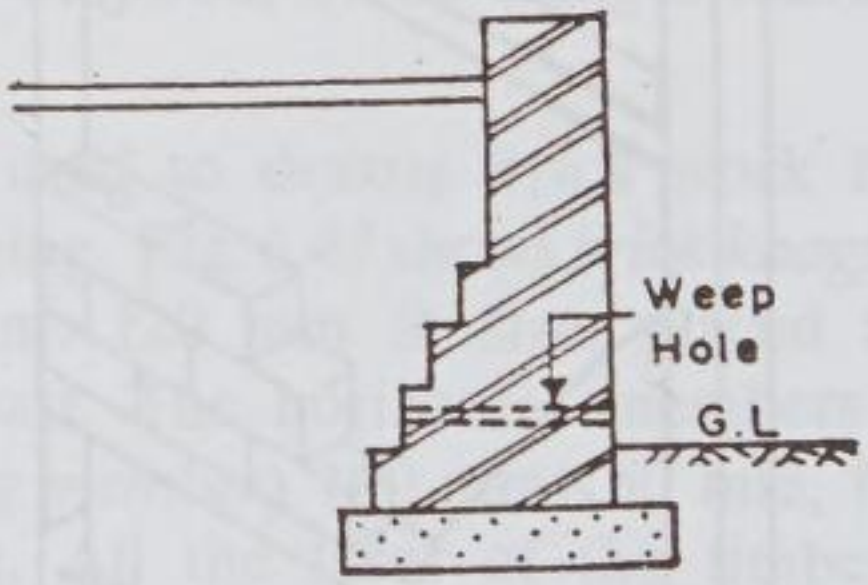
protect natural sloping ground from the cutting action of weathering agents. Fig. 6.48 (*a*) shows a retaining wall and a breast wall in respective positions. The method of designing both the walls is the same; only the function of each is different. The following salient points are note worthy :

1. Because of the increase of earth pressure with the depth of fill, the section of retaining wall/breast wall increases from top to bottom. Generally, the back of the wall is stepped while the face is kept either vertical or inclined.

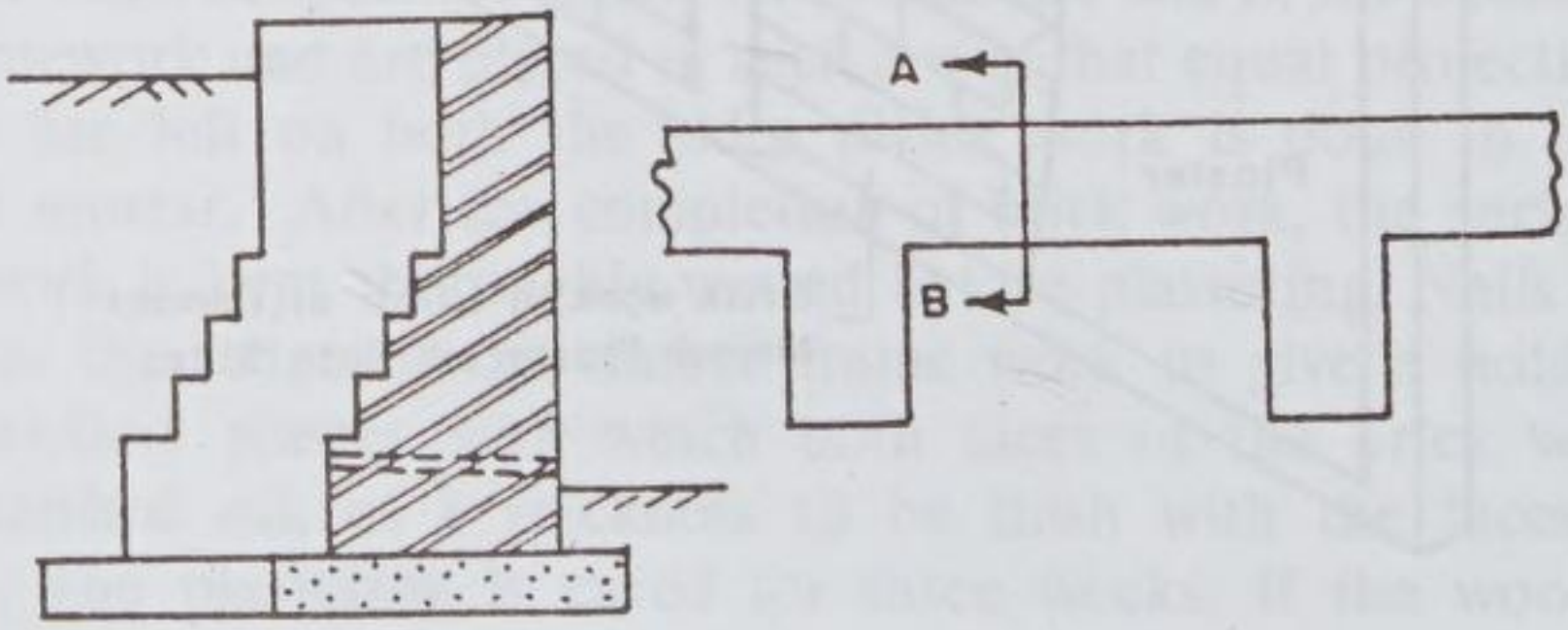




(b). Breast Wall



(c) Retaining Wall



Section A B Plan at Top

(d) Counterfort Retaining Wall

FIG. 6.48.

2. Breast walls are some times provided with batter on both sides, as shown in Fig. 6.48 (*b*).

3. When the height of fill is large, simple retaining walls become uneconomical. In that case, lateral supports are provided on earth side, at regular interval (3 to 4 metres). Such a wall is known as *counterfort retaining wall,* as shown in Fig. 6.48 (*d*). The counterforts must be tied to the main wall with internal iron ties to counteract any tendency to fracture at the junction.

4. As a thumb rule, the thickness of wall at any depth h before the fill level, may be kept between 0.33 h to 0.4 h, depending upon the conditions of the filled material.

5. The stability of retaining wall should be checked against sliding and overturning. In addition to this, the resultant of the vertical load and horizontal earth pressure, should fall within the middle third of the base, so that tension does not develop. The maximum compressive stress at the bottom level of brick course should not exceed the safe compressive stress for brick masonry. The maximum compressive stress below the concrete base should not exceed the safe bearing capacity of soil.

6. Sufficient number of weep holes should be provided all along the length as well as the height of the wall, to drain off gravitational water of the earth fill. General rule is to provide at least one weep hole for 3 square metre of the surface.

PROBLEMS

1. Compare brick and stone masonry.
2. (*a*) What do you understand by modular bricks ?
   (*b*) Draw sketches for the following bricks :
   (*i*) Bull nose brick
   (*ii*) Cant brick.
   (*iii*) Plinth header and plinth stretcher.
3. (*a*) Show with the help of sketches various types of closer bricks.
   (*b*) Show with the help of sketches various types of brick bats.
4. Write short notes on :
   (a) Header bond
   (*b*) Stretcher bond
   (*c*) Dutch bond
   (*d*) Garden Wall bond.
5. Differentiate and compare English bond, Flemish bond and double flemish bond.

6. Draw plans of alternate courses of (*i*) $1\frac{1}{2}$ brick wall, and (*ii*) 2-brick thick wall in (*a*) English bond (*b*) Double flemish bond.

7. Explain the method of providing bond at T-junction of two walls in (*i*) English bond, (*ii*) Double Flemish bond, for (*a*) $1\frac{1}{2}$ brick thick external and internal walls, and (*b*) 2-brick thick external wall and $1\frac{1}{2}$ brick thick internal wall.

8. Draw the plan of alternate courses in English bond for cross- junction of two walls of $1\frac{1}{2}$brick thickness.

9. What do you understand by a squint junction ? Draw typical sketches showing squint junction in (*a*) English bond (*b*) Double Flemish Bond.

10. Sketch the alternate courses of a $2\frac{1}{2}$ brick pier in (*a*) English bond (*b*) double Flemish bond.

11. What do you understand by attached piers ? Draw typical sketches in English and double Flemish Bonds.

12. Draw typical sketches of alternate courses, showing bond in brick footing of a pier.

13. Explain the modern method of laying the bricks. What special gadgets do you use ?

14. Write important points connected with the supervision of brick work.

15. Write a note on various defects in brick work.

16. (*a*) Explain the factors that affect the strength of brick masonry.
(*b*) How do you decide the thickness of a brick wall ?

17. Write short notes on the following :
(*a*) Brick buttresses.
(*b*) Brick corbel.
(*c*) Brick coping.
(*d*) Thresholds.
(*e*) Brick jambs.

18. What do you understand by brick knogging ? Explain the method of construction, with a neat sketch.

7

Masonry-3
Composite Masonry

7.1. INTRODUCTION

Composite masonry is the one which is constructed out of two or more types of building units or of different types of building materials. The composite masonry may be adopted due to two reasons :

(*i*) Improvement in the appearance of walls, etc.,

(*ii*) Use of available materials, to obtain optimum economy.

Composite masonry may be of the following types :

1. Stone-composite masonry.
2. Brick stone composite masonry.
3. Cement concrete masonry.
4. Hollow clay tile masonry.
5. Reinforced brick masonry.
6. Glass block masonry.

7.2. STONE COMPOSITE MASONRY

Composite stone masonry generally, consists of a combination of ashlar masonry and rubble masonry. Rubble masonry is generally very cheap, while ashlar masonry gives pleasing appearance. Hence rubble masonry is used in backing of the wall while the ashlar masonry is used in the facing, as shown in Fig. 7.1.

In order that both the facing and backing of the wall act monolithically, it is essential to observe utmost care during construction. The following points should be specifically attended to :

(1) Through stones should be used at regular interval, and in sufficient number.

(2) The backing and facing portions should be constructed in rich cement mortar.

(3) Construction of both the backing and facing should be carried out simultaneously so that proper bond is obtained.

(4) If necessary, metal cramps, dowels, lead plugs etc. should be provided between facing and backing.

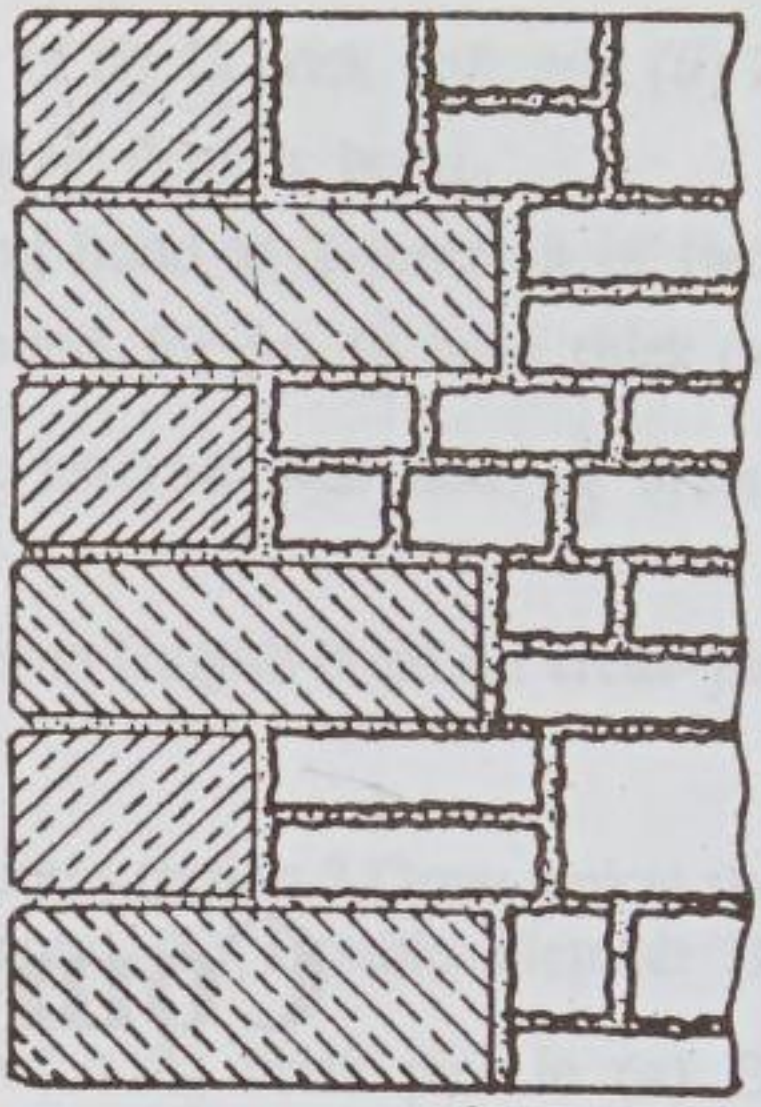


FIG. 7.1.

7.3. BRICK-STONE COMPOSITE MASONRY

Bricks and stones can be simultaneously used in three forms of composite masonry :

(*i*) Brick-backed ashlar masonry
(*ii*) Brick-backed stone slab facing
(*iii*) Rubble-backed brick masonry.

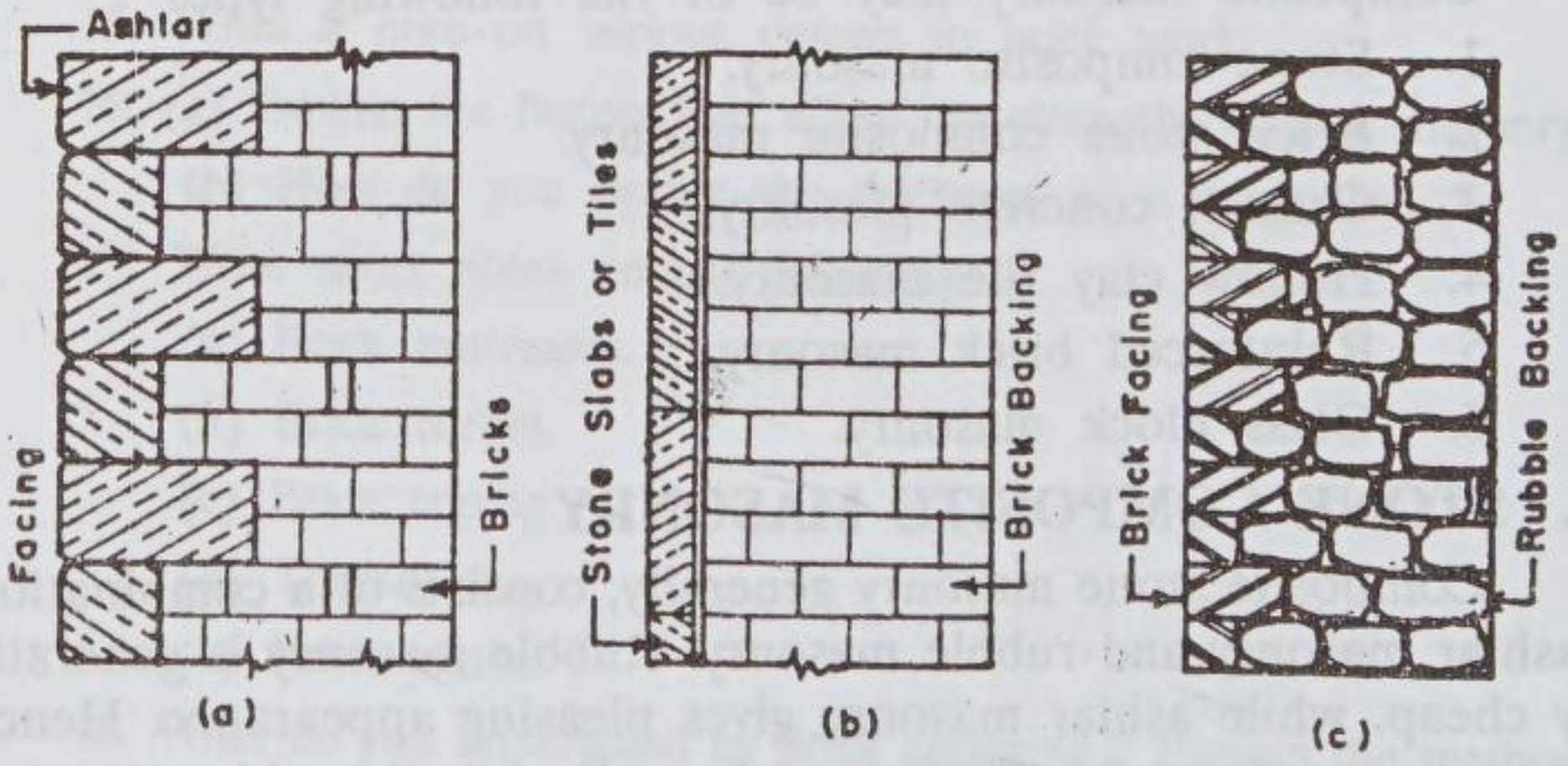


FIG. 7.2. BRICK-STONE COMPOSITE MASONRY.

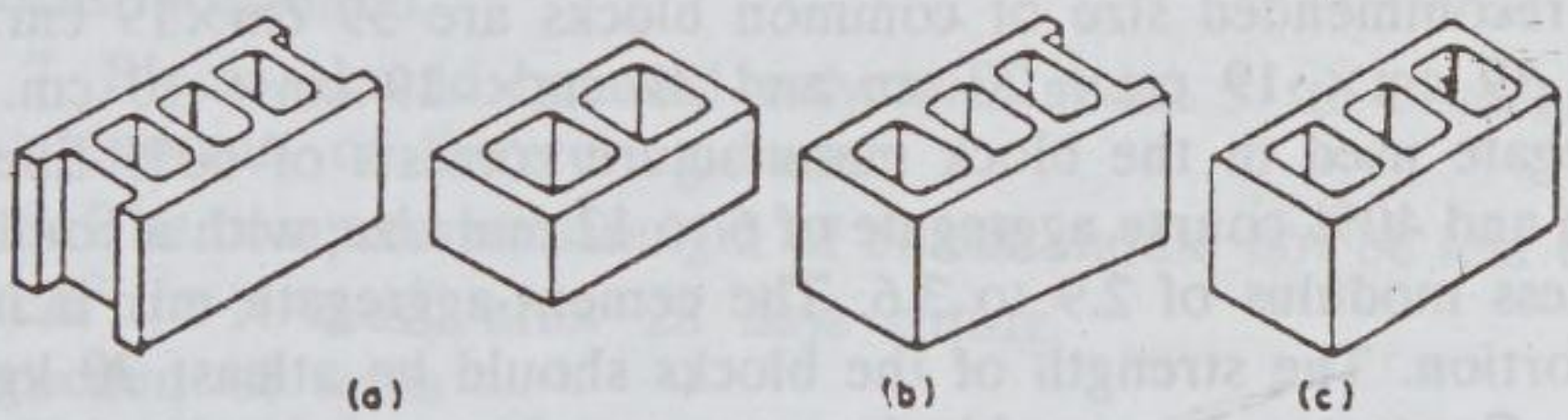
Fig. 7.2 (*a*) shows *brick-backed ashlar masonry.* The ashlar may be rough tooled. It is preferable to use the height of ashlar as a multiple of brick thickness plus masonry joints, so that coursed masonry is obtained. Cement mortar should be used for construction. Bricks should be laid in proper bond. Alternate courses of ashlar may be headers. Under each projecting course of ashlar, header bricks should be used.

Fig. 7.2 (*b*) shows the facing of stone slabs or stone tiles. The backing consists of bricks laid in courses with proper bond. This type of construction is quite common, since stone tiles may be of marble stone. If stone slabs are used, they are fine dressed, and are used in big panels. It is preferable to use metal cramps to connect the facing and backing masonry of the wall.

Fig. 7.2. (*c*) shows a rubble-backed brick masonry. It is commonly used at locations where rubble stone is available in large quantities, but ashlar is not available. In that case, the facing of the wall may be done in bricks laid in courses. Each alternate brick course consists of quoin header.

7.4. CONCRETE MASONRY

Concrete masonry or cement-concrete masonry uses cement concrete blocks, either hollow or solid, for wall construction, with or



(a) STRETCHER BLOCKS (b) CORNER BLOCK (c) DOUBLE CORNER OR PILLAR BLOCK (d) JAMB BLOCKS (e) PARTITION BLOCKS (f) SOLID BLOCK (g) BEAM OR LINTEL BLOCK (h) FLOOR BLOCK (i) FROGGED BRICK BLOCK AND (j) SOLID BRICK BLOCK (k) BULL NOSE BLOCK (l) LINTEL BLOCK

FIG. 7.3. CONCRETE MASONRY BLOCKS.

without stone facing. A *hollow unit,* is defined as that unit which has core-void area greater than 25% of the gross area. Various types of *concrete masonry units*, depending upon shape and size, are manufactured, and these can be grouped in two heads :

(*i*) Regular concrete blocks
(*ii*) Hollow concrete units.

Regular concrete blocks are manufactured from dense aggregate, and they are used in load bearing walls. *Hollow concrete units* are manufactured from light weight aggregates. They may be used both for load bearing as well as non-load bearing walls. They are light in weight. Fig. 7.3 shows various forms of concrete masonry units.

Concrete Association of India recommends that the face thickness of the hollow blocks should atleast be 5 cm, and the net area should atleast be 55 to 60% of the gross area. The cores in the blocks should atleast be two in number and should preferably be oval shaped. The recommended size of common blocks are 39 cm×19 cm× 30 cm ; 39 cm× 19 cm× 20 cm and 39 cm× 19 cm× 10 cm. The aggregate used in the block manufacture consists of 60% fine (*i.e.* sand) and 40% course aggregate of 6 to 12 mm size, with a combined fineness modulus of 2.9 to 3.6. The cement-aggregate mix is in 1:6 proportion. The strength of the blocks should be atleast 30 kg/cm^2.

Concrete masonry blocks are manufactured in the following *surface finishes:*

(*i*) Common finished surface.
(*ii*) Glazed finish.
(*iii*) Slumped finish.
(*iv*) Specially faced finish.
(*v*) Coloured finish.

Common finish surface has fine to course texture which can be obtained by varying the mix proportions and by using appropriate aggregates. If the exposure of the aggregates is required, it can be obtained either by treating the surface by dilute acid solution or by scrubbing it while the concrete has not fully set. *Glazed finish* is used for decorative work. It can be obtained in a manner similar to glazing of tiles. Glazed finish concrete blocks are water resistant. *Slumped finish* is the *rough finish* which is obtained by using the concrete of desired slump. When the forms are open, the blocks settle slightly, causing rough surface. In *specially faced finish,* finishing material such as marble etc. is incorporated on the facing side of the block. *Coloured finish* can be obtained by mixing various pigments to the concrete mix.

Manufacture of concrete masonry blocks

The following points should be kept in mind while manufacturing the concrete masonry bricks :

1. The cement-aggregate ratio should not be leaner than 1 : 6.

2. The aggregate should have a mixture of fine aggregate 60% and coarse aggregate (6 to 12 mm size) 40%. The fineness modulus as the mixed aggregate should be between 2.9 to 3.6.

3. Blocks should be taken out from the moulds only when concrete has sufficiently set.

4. Concrete should not have very lean consistency. If hand moulding is done, the hollows should be vertical. Proper compaction should be obtained.

5. Machine casting is preferable to hand casting, to obtain better finish.

6. After taking the blocks out of mould, they should be kept under shade for atleast 24 hours, and then immersed in water tank for curing for atleast one week. After that, the blocks may be stacked with cells horizontal.

7. Blocks should be used only *after* about 3 to 4 weeks of their taking out of the curing tank.

8. The compressive strength of blocks should not be less than 30 kg/cm^2 (3 N/mm^2) after 28 days curing.

Construction of walls

The method of constructing the wall with concrete blocks is the same as that used for brick masonry. First, the corners or ends of the wall are constructed with few courses of blocks. Mortar is applied to the bottom of the concrete block at the horizontal face members only. For vertical joints, the mortar is applied to the projections at the sides of the block. For building the portion in between the corners, the string is spread between the two horizontal end blocks of a course, and the blocks are laid in between. The final closing block is fitted carefully.

The following points should be kept in mind while supervising the construction work :

1. Before use, it should be ensured that the blocks are *dry*. They should not be drenched in water before use.

2. Blocks of successive courses should be so laid that vertical joints are staggered.

3. The joints should be 5 to 10 mm thick, and should be uniform.

4. The mortar used for construction should not be stronger than the concrete mix used for manufacture of blocks. Generally cement-lime-sand mortar of mix proportion 1 : 1 : 10 is used.

5. The blocks used for external walls should have absorption less than 10%. For internal walls, the absorption should be less than 15%.

6. Concrete blocks have high thermal expansion, due to which walls crack at corners. Long walls may have cracks even at its mid-length. Hence at the junction of walls, solid concrete blocks or hollow blocks filled with concrete should be used.

Wall thickness

Table 7.1. gives the thickness of walls made of hollow concrete blocks.

TABLE 7.1
WALLS THICKNESS (cm)

No. of floors	*Foundation or basement*	*Ground floor*	*1st floor*	*2nd floor*	*3rd floor*
—	20 to 30	20	—	—	—
1	30	20	20	—	—
2	30	30	30	20	—
3	40	40	30	30	20

Advantages of hollow concrete block masonry

1. Concrete blocks are regular in size, requiring no dressing work. Hence construction is very rapid.
2. Blocks are light and therefore easy to handle.
3. Because of their lightness, the loads transferred to foundations is much less than the stone masonry. This is important consideration in locations where soil has low hearing capacity.
4. There is great saving in the material.
5. Hollow blocks are structurally stronger than bricks.
6. Thinner walls can be easily constructed, resulting in increase in the floor area.
7. Because of large size of the blocks, the number of joints in the masonry is less. This results in saving in mortar.
8. Because of hollow space, the resulting wall has better insulating properties against sound, heat and dampness.
9. Blocks can withstand the atmospheric actions, and do not require plaster or any other covering or facia work.

7.5. HOLLOW CLAY BLOCKS MASONRY

Hollow clay blocks (or tiles) are made of selected clay or diatomaceous earth, which is dried and burned. The clay blocks are used to build foundations, walls, partitions, floors and other structural members. Even though the walls of the blocks are relatively thin, they are quite strong and light. These tiles are fire proof, resistant to termite and free from decay caused by the contact of moisture or chemicals. Because of large amount of air within the cells of

blocks, the thermal insulation is very good.

Hollow clay blocks are manufactured in various shapes and sizes. They are also made of various grades, such as load bearing (L.B.) and extra load bearing (L.B.X.). Fig. 7.4 shows various shapes and sizes of structural clay units. The shell of a clay block constitutes the four sides surrounding the hollow interior, while the webs serve as partitions between the cells. The overall average thickness of the shells should not be less than 2 cm and of the web not less than

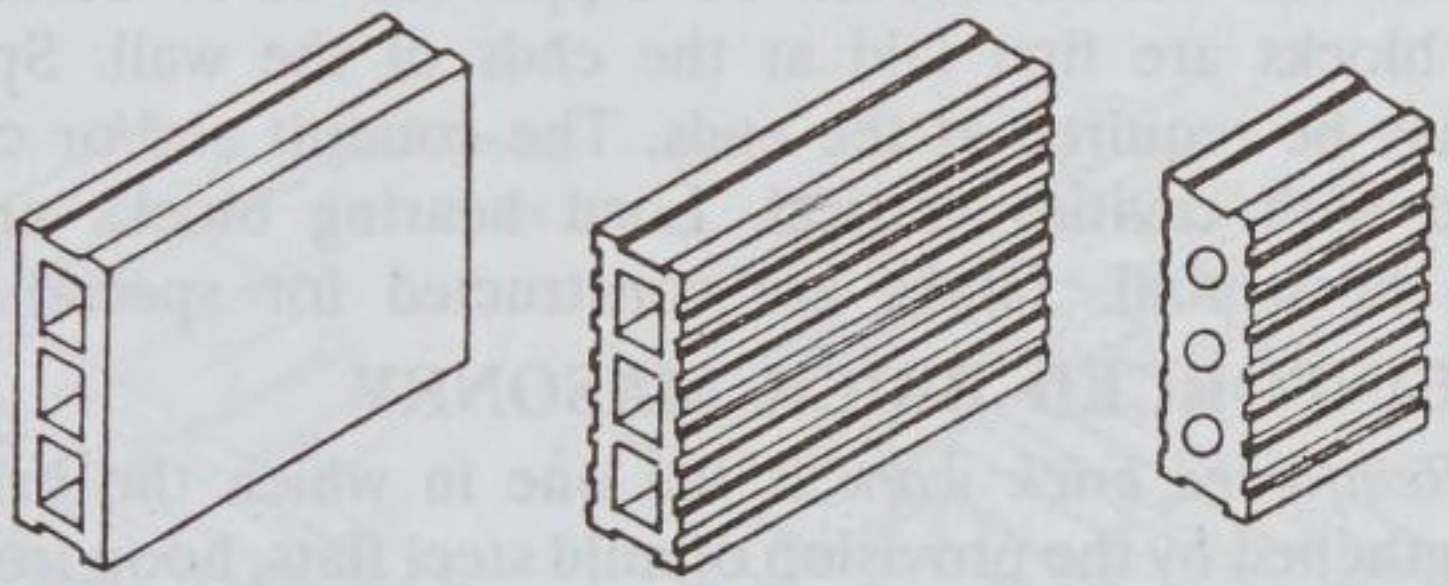


FIG. 7.4. CLAY BLOCK UNITS.

1 cm for end construction blocks. Tiles may have *grooves* on one or more faces. The area covered by grooves should not exceed 50 % of the area of cored faces. Grooved tiles are used only where plastering is to be done : otherwise smooth tiles should be used.

The load bearing main walls and partition walls should be constructed in 1 : 1 : 6 (cement, lime, sand), and non-load bearing main walls and partition walls are generally constructed in 1 : 2 : 9 mix.

All the blocks should be dipped in water before use. The corner blocks are first laid at the ends of the wall. Special closer units may be required at the ends. The conduit and/or closer blocks are laid with cavities vertical. Load bearing blocks are laid with cavities horizontal. Jambs are constructed for special blocks.

7.6. REINFORCED BRICK MASONRY

Reinforced brick work is the one in which the brick masonry is strengthened by the provision of mild steel flats, hoop iron, expanded mesh or bars. It is adopted or used in the following circumstances:

1. When the brick work has to bear tensile and shear stresses.
2. When it is required to increase the longitudinal bond.
3. When the brick work is supported on soil which is susceptible to large settlement.
4. When the brick work is supposed to act as a beam or lintel over openings.
5. When the brick work is to resist lateral loads, such as in retaining walls etc.
6. When the brick wall is to carry heavy compressive loads.
7. When the brick work is to be used in seismic areas, since it can also resist lateral loads.

Reinforced brick work uses first class bricks with high compressive strength. Dense cement mortar is used to embed the reinforcement. The reinforcing material may be (*i*) hoop iron, (*ii*) mild steel bars, (*iii*) mild steel flats and (*iv*) expanded mesh. The reinforcement is laid either horizontally or vertically.

(*a*) Horizontal reinforcement

Horizontal reinforcement *for wall* consists of either (*i*) wrought iron flat bars, known as *hoop iron,* or (*ii*) steel mesh. Fig. 7.5 (*a*) shows the hoop iron reinforcement for a brick wall. Generally, two strips of hoop iron are used per header brick and one hoop iron per stretcher brick *i.e.*., one strand of hoop iron for each half brick

thickness of wall. Mild steel flats may also be used in place of hoop iron. It is usual to reinforce *every sixth course.* Mild steel flat bars may have width between 22 to 32 mm and thickness equal to 0.25 to 1.6 mm. Protection against rust is provided by dipping the bars in hot tar; these are then at once sanded to increase the adhesion of the mortar. At the ends (quoins), the bars are beaten flat and then double hooked to bars coming from transverse direction. At

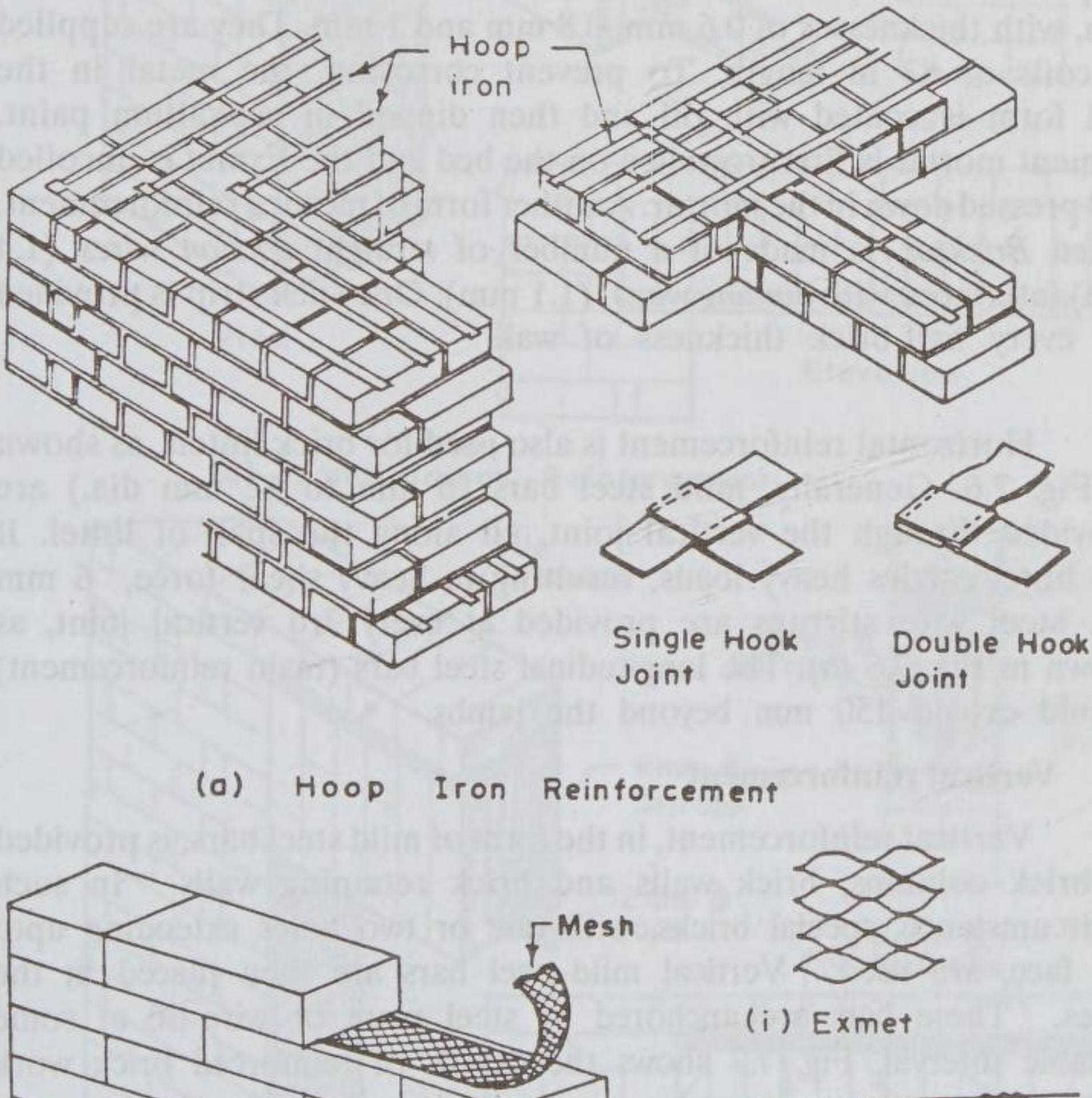


FIG. 7.5. HORIZONTAL REINFORCEMENT IN WALLS.

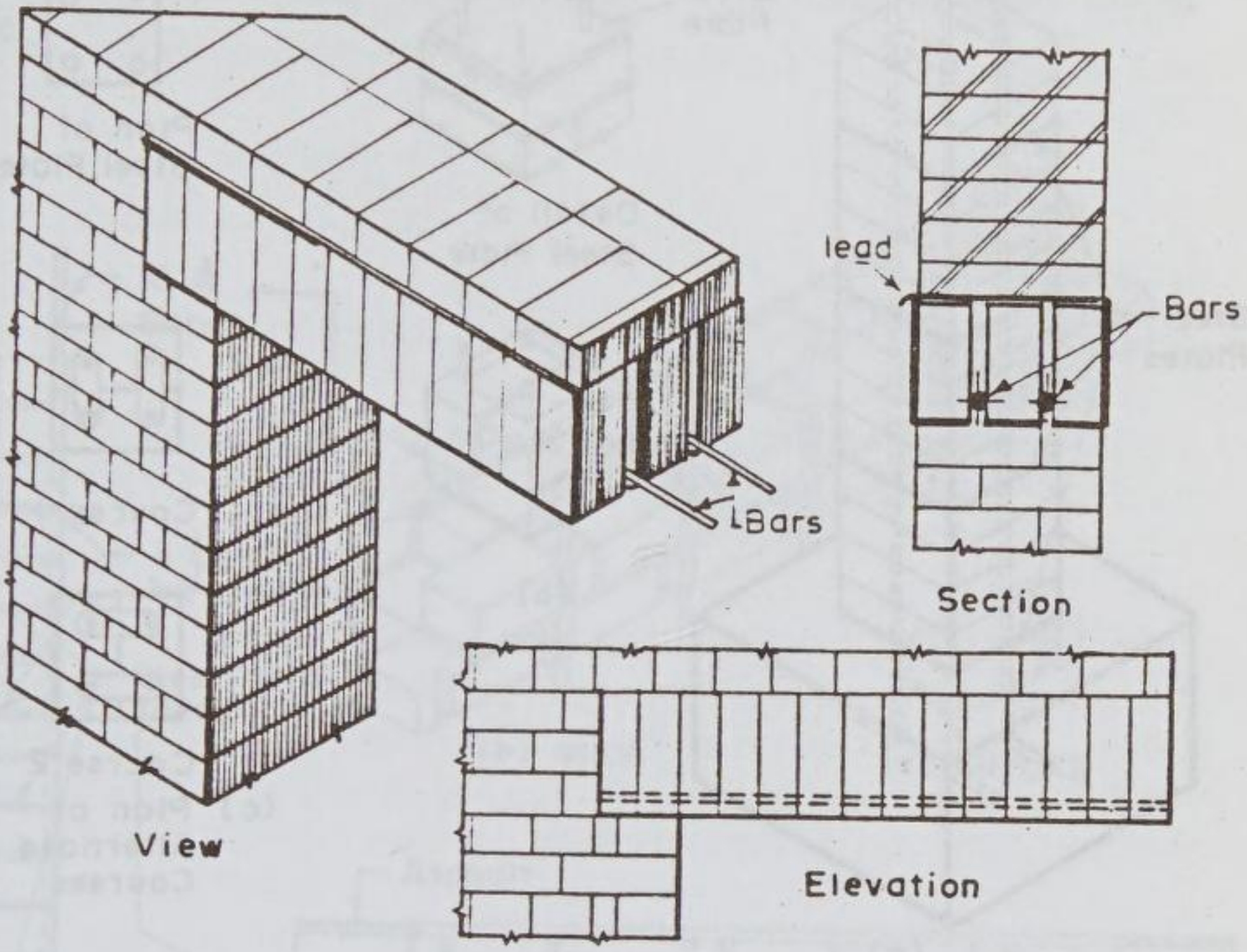
the junctions, the bars crossing each other are interlaced and single hooked. Hoop iron is now rarely used because of its higher cost and because of its thickness, unless thicker joints are used.

Another form of horizontal reinforcement, which is more commonly used, is the provision of steel meshed strips called *Exmet,* made from their rolled steel plates which are cut and stretched (or expanded) by a machine to diamond network. Such a strip is known as *expanded metal* (*Exmet*) and is provided at *every third course.* These strips are available in widths of 65 mm, 178 mm and 230 to 305 mm, with thicknesses of 0.6 mm, 0.8 mm and 1 mm. They are supplied in coils of 83 m length. To prevent corrosion, the metal in the coil form is coated with oil and then dipped in asphaltum paint. Cement mortar is first trowelled on the bed and the Exmet is uncoiled and pressed down in the mortar. Another form of meshed reinforcement, called *Bricktor,* is made of a number of straight *tension wires* (1.4 mm) interlaced with *binding wires* (1.1 mm). One such strip is provided for every half-brick thickness of wall.

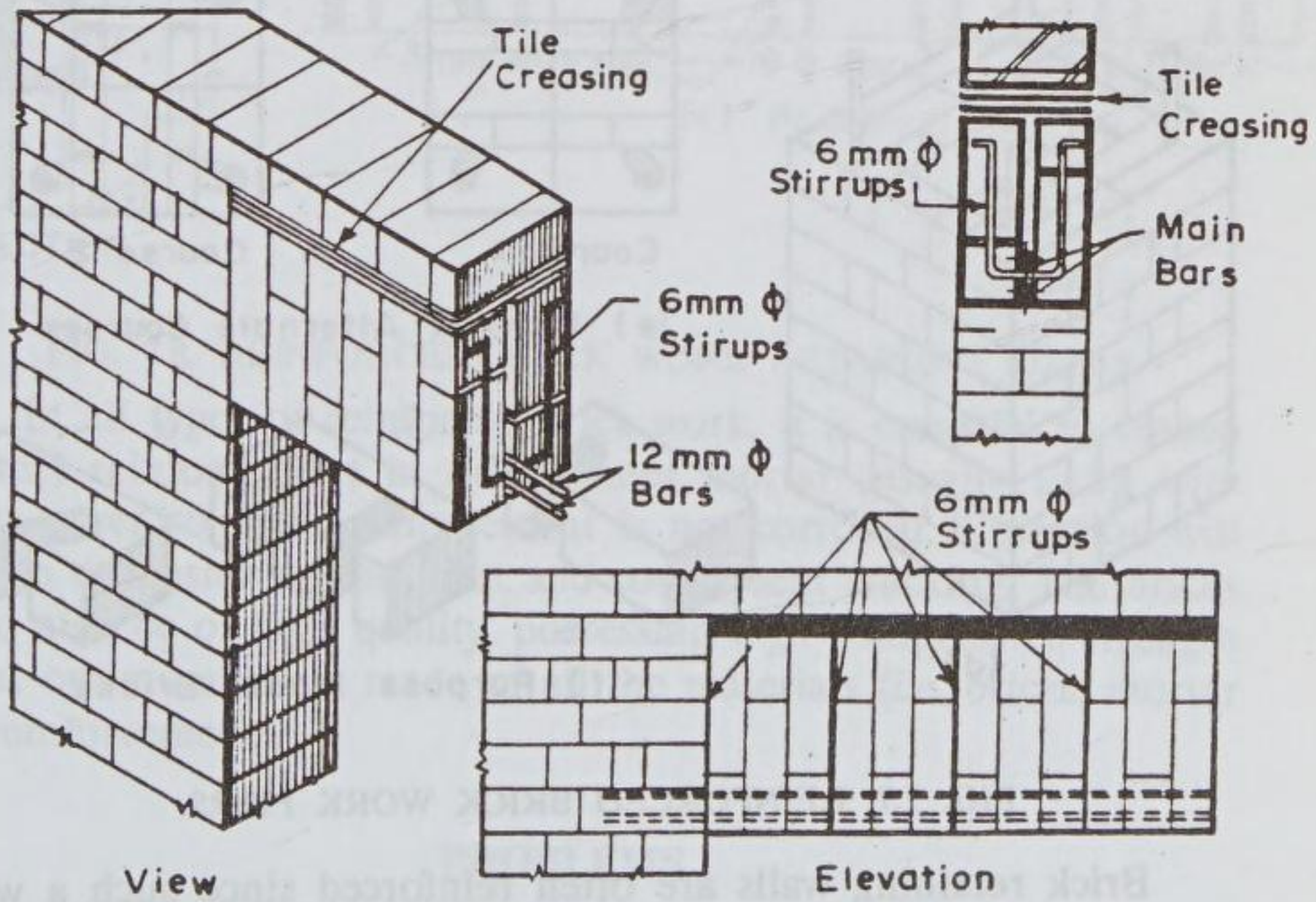
Horizontal reinforcement is also used for brick lintels, as shown in Fig. 7.6. Generally, mild steel bars (6 mm to 12 mm dia.) are provided through the vertical joint, all along the span of lintel. If the lintel carries heavy loads, resulting in heavy shear force, 6 mm dia. steel wire stirrups are provided at every 3rd vertical joint, as shown in Fig. 7.6 (*b*). The longitudinal steel bars (main reinforcement) should extend 150 mm beyond the jambs.

(*b*) Vertical reinforcement

Vertical reinforcement, in the form of mild steel bars, is provided in brick columns, brick walls and brick retaining walls. In such a circumstance, special bricks, with one or two holes extending upto the face, are used. Vertical mild-steel bars are then placed in the holes. These bars are anchored by steel plate or wire-tie at some suitable interval. Fig. 7.7 shows the details of reinforced brick work piers.

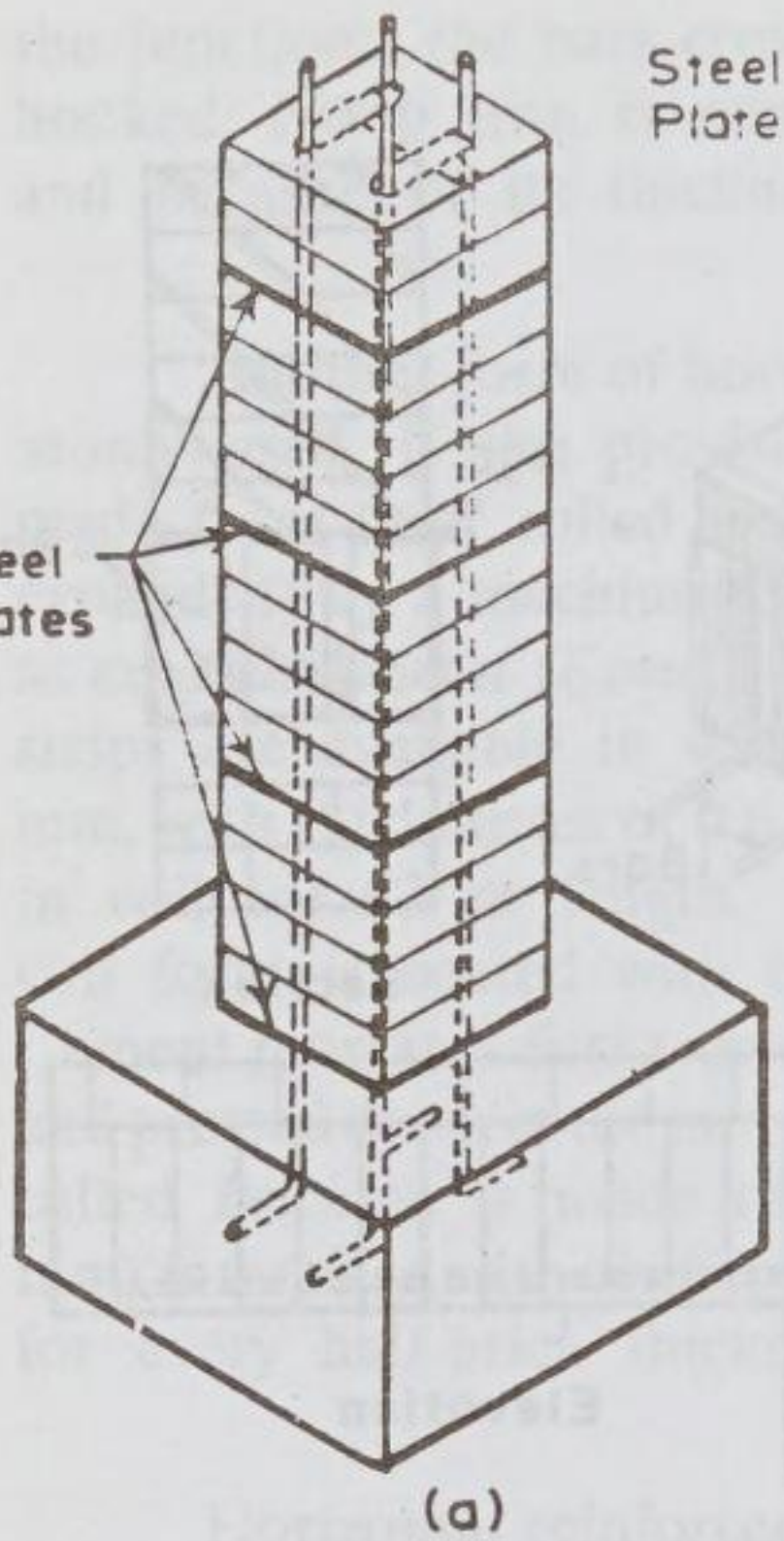


(a) Longitudinal Reinforcement

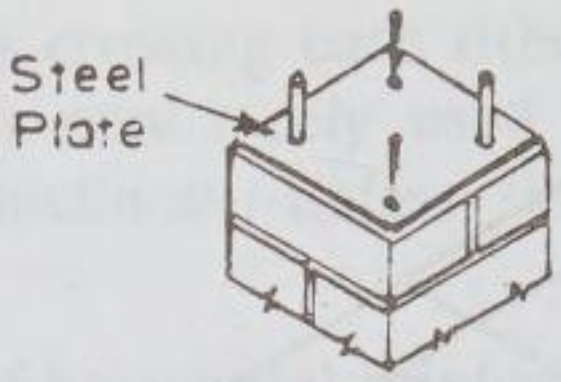


(b) Longitudinal Reinforcement with Stirrups

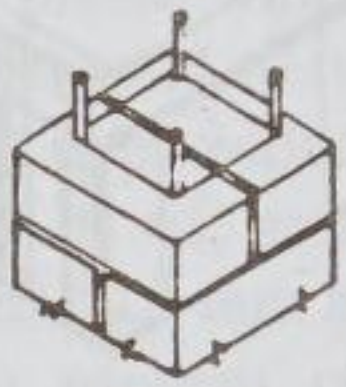
FIG. 7.6. REINFORCED BRICK WORK LINTELS.



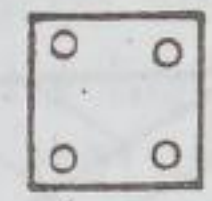
(a)



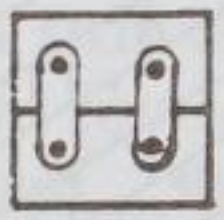
Detail of Steel Plate



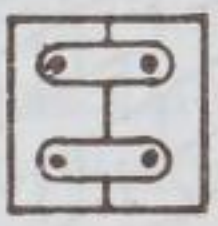
(b)



Plan of Steel Plate

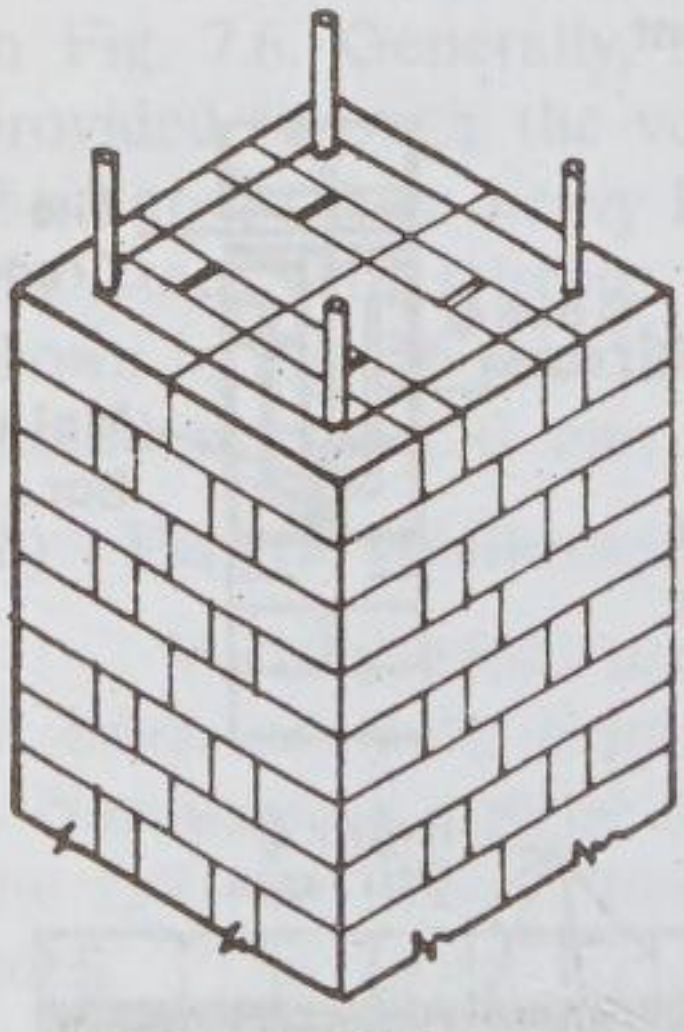


Course 1

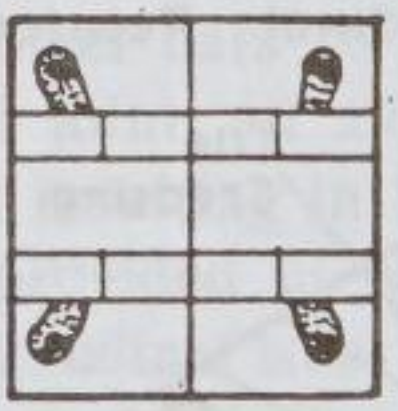


Course 2

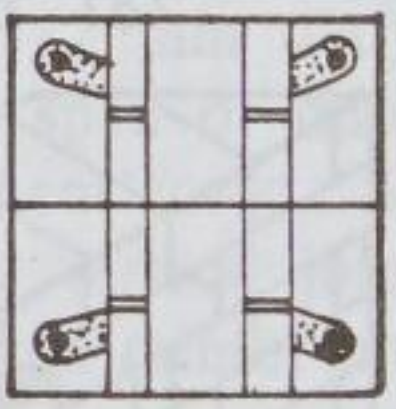
(c) Plan of Alternate Courses



(d)

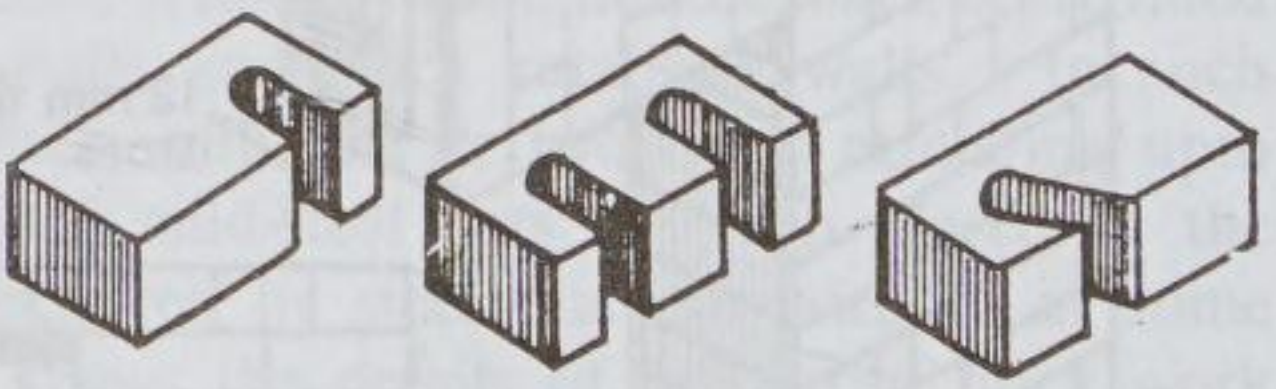


Course 1



Course 2

(e) Plans of Alternate Courses



(f) Purpose Made Bricks

FIG. 7.7. REINFORCED BRICK WORK PIERS.

Brick retaining walls are often reinforced since such a work is cheaper than the reinforced cement concrete, when the height of the wall is upto 3 m. Vertical reinforcing bars are placed vertically near each face, in addition to steel meshed strips at every fourth course. The bricks opposite each bar are purpose made, having a groove. The size of the groove is kept slightly more than the diameter

of the bar so that it may be grouted in with cement mortar, to prevent corrosion. Steel wire ties may be provided at every fourth course.

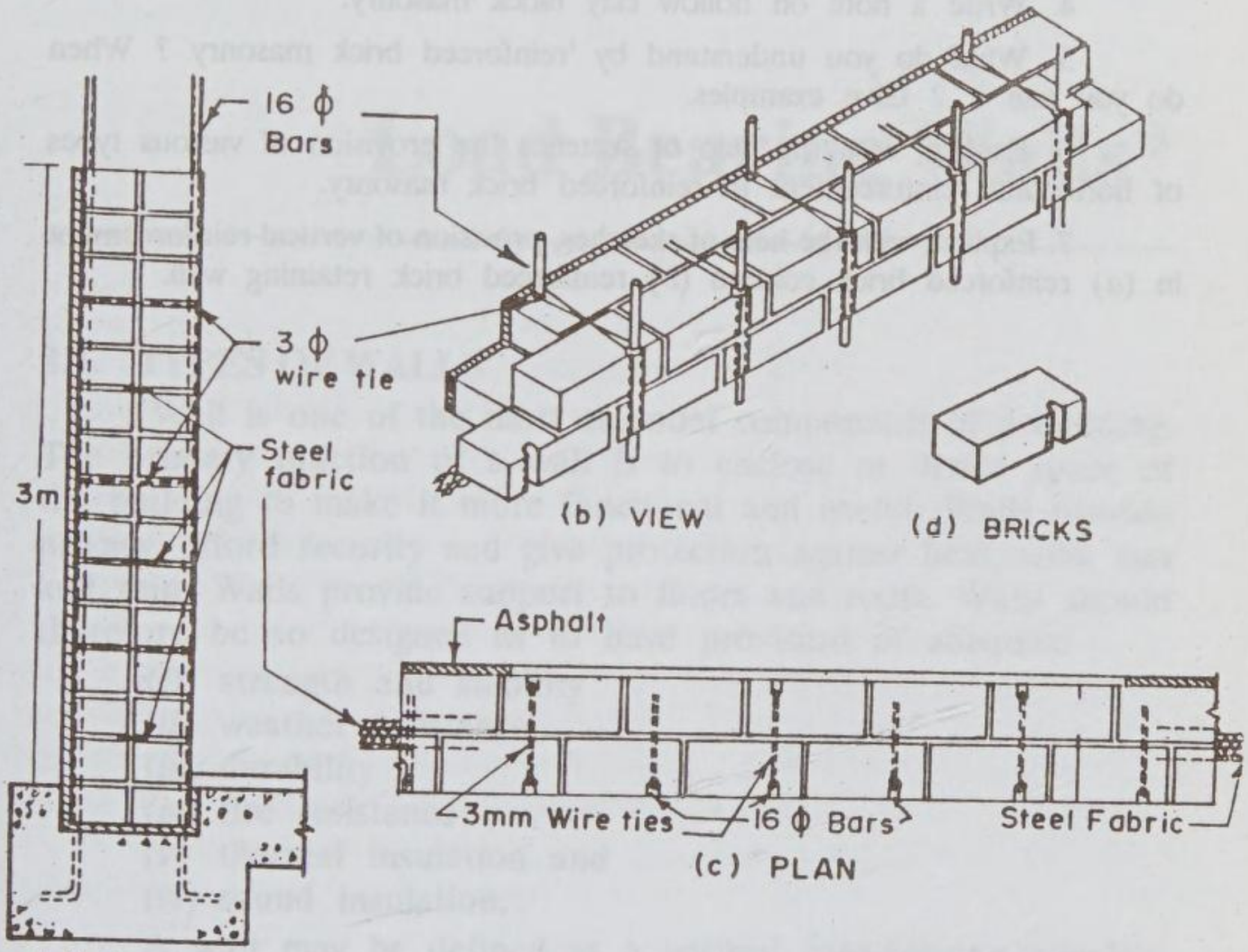


FIG. 7.8. REINFORCED BRICK WORK RETAINING WALLS.

In all types of reinforced brick work, it is essential to embed the steel reinforcement in rich cement mortar (usually 1 : 3), with proper cover so that reinforcement is not corroded. Corrosion will result in expansion of the joint and consequent cracking. The bricks should also be of high quality, possessing high compressive strength so that optimum use is made of all the materials (*i.e.* bricks, mortar and reinforcement).

PROBLEMS

1. What do you understand by 'composite masonry' ? Enumerate various types of composite masonry, and state the circumstances under which each type is used.

2. Describe, with the help of sketches, various forms of stone brick

composite masonry.

3. What do you understand by concrete masonry ? State the advantages of hollow block concrete masonry. State various types of surface finishes in such a masonry.

4. Write a note on hollow clay block masonry.

5. What do you understand by 'reinforced brick masonry ? When do you use it ? Give examples.

6. Explain, with the help of sketches the provision of various types of horizontal reinforcement in reinforced brick masonry.

7. Explain, with the help of sketches, provision of vertical reinforcement in (*a*) reinforced brick column (*b*) reinforced brick retaining wall.

8

Load Bearing Walls*

8.1. TYPES OF WALLS

Wall is one of the most essential components of a building. The primary function of a wall is to enclose or divide space of the building to make it more functional and useful. Walls provide privacy, afford security and give protection against heat, cold, sun and rain. Walls provide support to floors and roofs. Walls should therefore be so designed as to have provision of adequate

(*i*) strength and stability
(*ii*) weather resistance
(*iii*) durability
(*iv*) fire resistance
(*v*) thermal insulation and
(*vi*) sound insulation.

A wall may be defined as a vertical load-bearing member, the width (*i.e.*., length) of which exceeds four times the thickness. In contrast to this a *column* is an isolated load-bearing member, the width of which does not exceed four times the thickness.

Walls may be basically divided into two types :

(*a*) Load-bearing, and
(*b*) Non-load bearing.

Each type may further be divided into *external* (or enclosing) walls and *internal* or divide walls.

Load-bearing walls are those which are designed to carry super–imposed loads (transferred through roofs etc.), in addition to their own weight (self weight). Non-load-bearing walls carry their own-load only. They generally serve as *divide walls or partition*

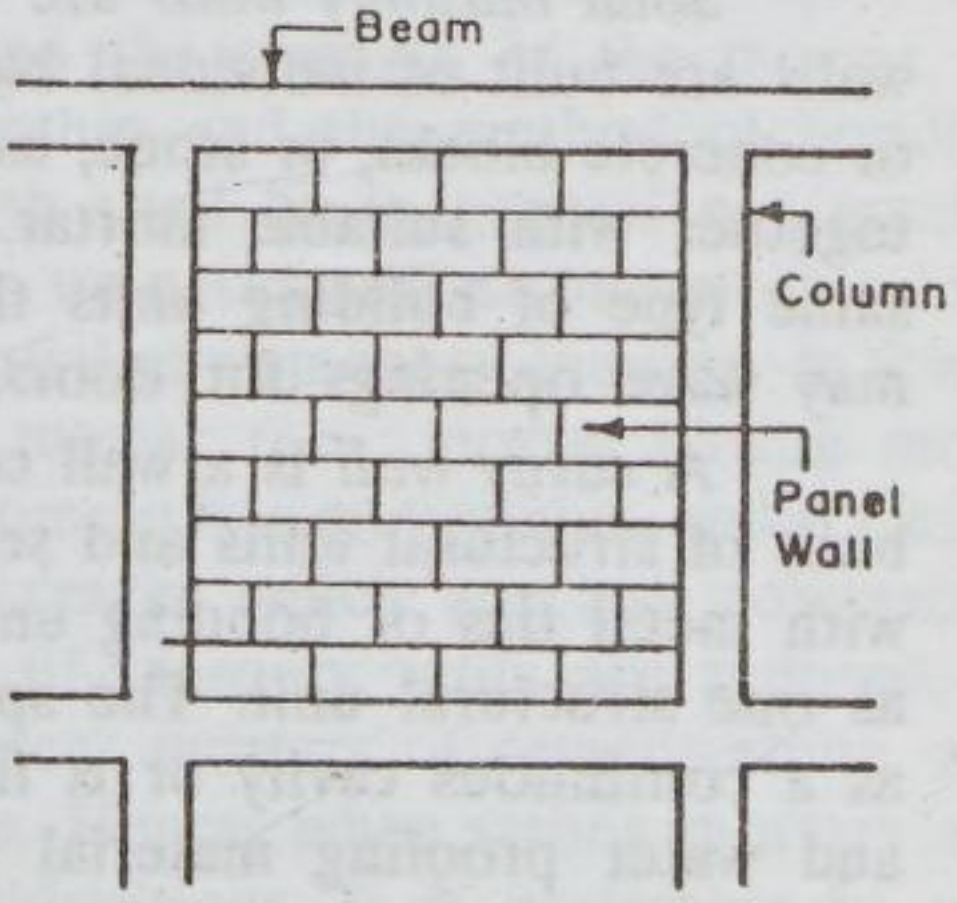


FIG. 8.1.(*a*) PANEL WALL.

*Junior students may omit this chapter.

walls. The external non–load–bearing wall, commonly related to framed structures is termed as *panel wall* (Fig. 8.1).

A *partition wall* is a thin internal wall which is constructed to divide the space within the building into rooms or areas. It may either be non-load-bearing or load bearing. A load-bearing partition wall is called an internal wall.

A *party wall* is a wall separating adjoining buildings belonging to different owners or occupied by different persons. It may, or may not, be load-bearing.

A *separating wall* is a wall separating different occupancies within the same building.

A *curtain wall* is a self-supporting wall carrying no other vertical loads but subject to lateral loads. It may be laterally supported by vertical or horizontal structural members where necessary [Fig. 8.1 (*b*)].

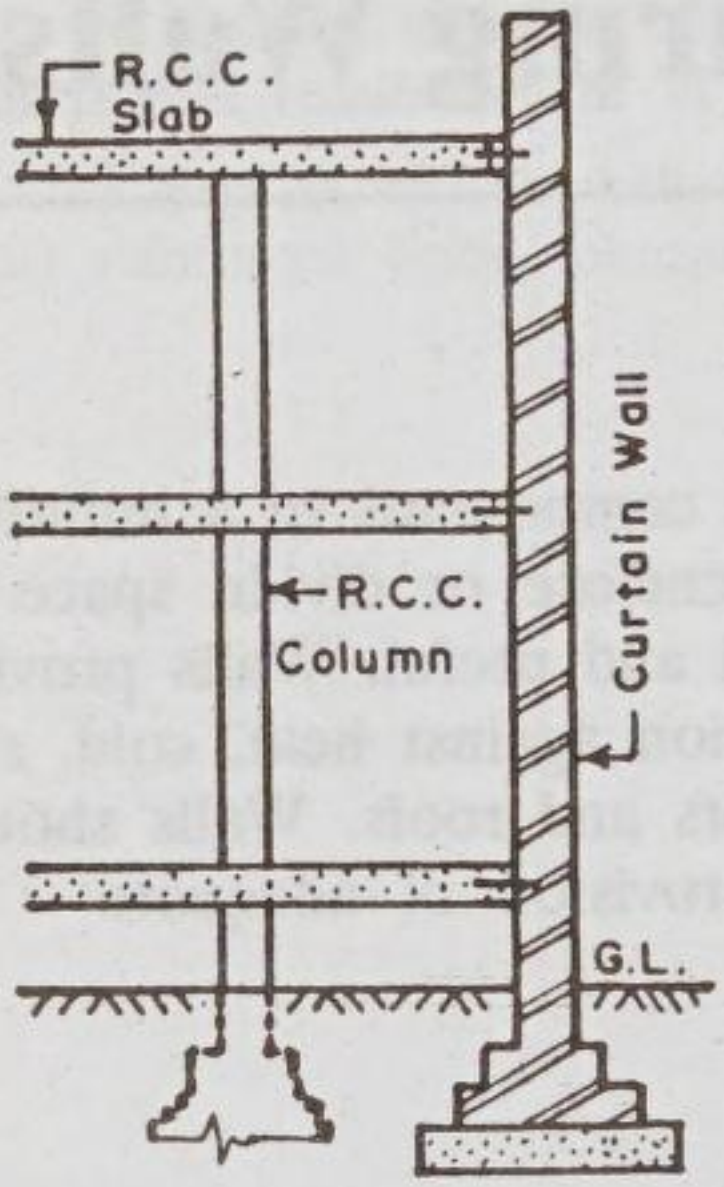


FIG. 8.1. (*b*) CURTAIN WALL.

Cross-wall construction is a particular form of load-bearing wall construction in which all the loads are carried by internal walls, running at right angles to the length of the building.

Load bearing walls may further be divided into the following types :

(*a*) Solid masonry wall
(*b*) Cavity wall
(*c*) Faced wall
(*d*) Veneered wall.

Solid masonry walls are the one most commonly used. These walls are built of individual blocks of material, such as bricks, clay or concrete blocks, or stone, usually in horizontal courses, cemented together with suitable mortar. A solid wall is constructed of the same type of building units throughout its thickness. However, it may have openings for doors, windows etc.

A *cavity wall* is a wall comprising two leaves, each leaf being built of structural units and separated by a cavity and tied together with metal ties or bonding units to ensure that the two leaves act as one structural unit. The space between the leaves is either left as a continuous cavity or is filled with non-load-bearing insulating and water proofing material (See Fig. 9.1).

A *faced wall* is a wall in which the facing and backing are of two different materials which are bonded together to ensure common

action under load (See Fig. 7.2).

A *veneered wall* is a wall in which the facing is attached to the backing but not so bonded as to result in a common action under load.

8.2. DESIGN CONSIDERATIONS

Load-bearing walls may be subjected to a variety of loads, *viz.*, live loads (super–imposed loads), dead loads, wind pressure, earthquake forces etc. Live loads and dead loads act in vertical direction. When the floor slabs transferring the loads to the wall are not supported through the full width of the wall, the loads act eccentrically, causing moments in the wall.

Load-bearing walls are structurally efficient when the load is uniformly distributed and when the structure is so planned that eccentricity of loading on the wall is as small as possible. The *strength* of a wall is measured in terms of its resistance to the stresses set up in it by its own weight, by super–imposed loads and by lateral pressure such as wind, etc .; its stability by its resistance to overturning by lateral forces and bucking caused by excessive slenderness.

In order to ensure uniformity of loading, openings in walls should not be too large and these should be, as far as possible, of 'hole in wall' type; bearings for lintels and bed blocks under beams should be liberal in size ; heavy concentration of loads should be avoided by judicious planning and sections of load-bearing members should be varied with the loadings so as to obtain more or less uniform stresses in adjoining parts of members. One of the commonly occurring causes of cracks in masonry is wide variation in stress in masonry in adjoining parts. Eccentricity of loading on walls should be reduced by providing adequate bearing of floors/roofs on the walls and making than as rigid as possible consistent with economy and other considerations.

The *strength* of a masonry wall depends primarily upon the strength of the masonry units and the strength of the mortar. In addition, the quality of workmanship and the method of bonding is also important. Mortar strength shall be in general not greater than that of the masonry unit. An un-necessarily strong mortar concentrates the effect of any differential movement of masonry in fewer and wider cracks while a weak mortar (*i.e..*, mortar having more of lime and less of cement) will accommodate movements, and cracking will be distributed as thin hair cracks which are less noticeable. Also, stresses due to expansion of masonry units are reduced, if a week mortar is used. Lean cement mortars of cement alone, are harsh, pervious and less workable. Hence, when strong mortars are not required from strength considerations, it is preferable to use composite mortars of cement, lime and sand in appropriate proportions.

However, rich cement mortar is needed : (*a*) When masonry units of high strength are used so as to get strong masonry, (*b*) when early strength is necessary for working under frosty conditions, and (*c*) when masonry is in wet location as in foundation below plinth, where a dense mortar being less pervious can better resist the effect of soluble salts.

The *thickness* of a load-bearing wall should be sufficient at all points to ensure that the stresses due to the worst conditions of loading for which the structure is designed are within the limits prescribed for that particular type of wall. The thickness used for design calculations should be the *actual thickness* of the masonry and not the *nominal thickness*. In the case of modular bricks, thickness of one brick wall will be 19 cm actual and 20 cm nominal. Similarly, the thickness of $1\frac{1}{2}$ brick wall will be 29 cm actual and 30 cm nominal (See Fig. 8.2 *a*). Thus,the actual thickness is computed as the sum of the average dimensions of masonry units together with the specified joint thickness. If joints are raked to provide key for subsequent plastering, the thickness should be reduced by the depth of the raking out. Thus, in Fig. 8.2 (*b*) the joints in one side is raked to a depth of 1 cm, and hence the effective thickness of wall =19−1=18 cm.

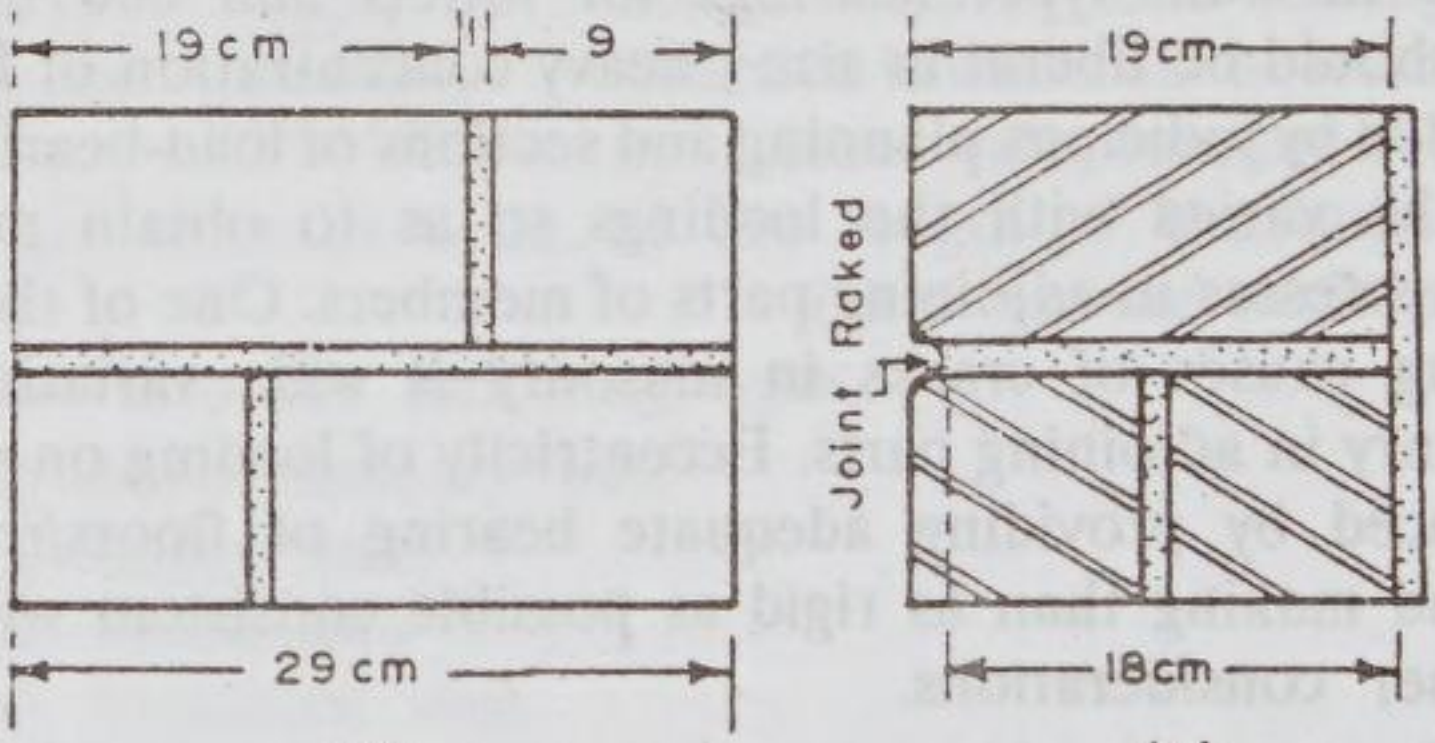


FIG. 8.2. THICKNESS OF WALL.

When vertical loads act on the wall, either axially or at small eccentricity, the wall behaves like a column. Its strength, of the same vertical load intensity, depends upon the *slenderness ratio* which is a function of (*i*) height of the wall, and (*ii*) length of the wall, and (*iii*) thickness of wall, and (*iv*) support conditions. The *slenderness ratio* of a wall is the ratio of its effective height divided by the effective thickness or the effective length divided by the effective thickness, whichever is less. The effective height and effective length of the wall depend upon the lateral support to the wall.

8.3. LATERAL SUPPORT

A wall may be considered to be provided with *adequate lateral support* if the construction providing the support is capable of resisting

the sum of following lateral forces :

(*a*) The simple static reactions to the total applied horizontal forces at the point of lateral support, and

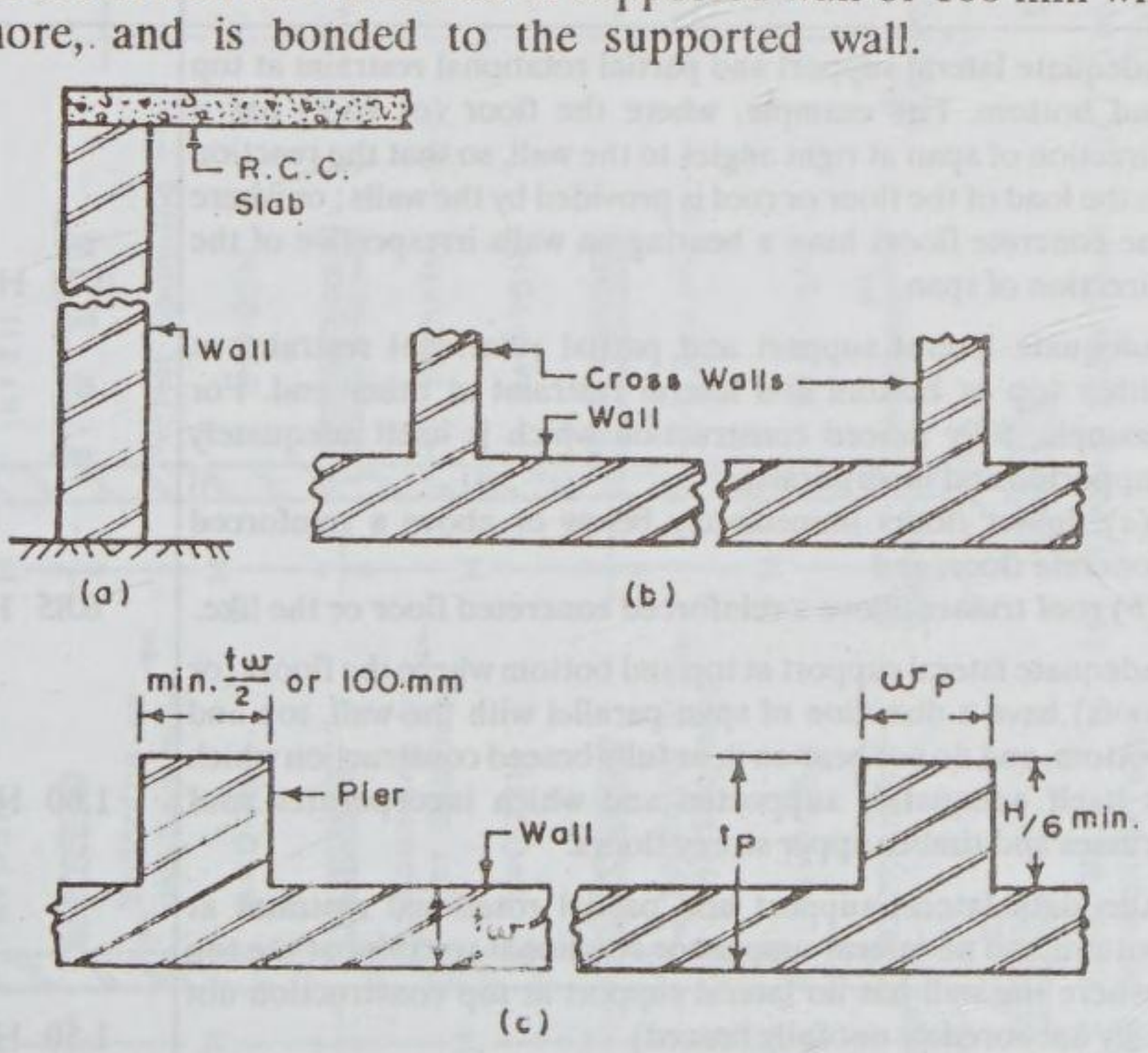
(*b*) Two and a half percent of the total vertical load that the wall is designed to carry at the point of lateral support. Lateral support to a wall has to perform two important functions, *i.e.*,

(*i*) to limit the slenderness so as to prevent buckling and (*ii*) to provide stability to the structure against over-turning on account of horizontal forces.

A wall can be laterally supported either at vertical intervals by floor roof transmitting horizontal forces to cross-walls and then to the foundation or at horizontal interval by cross–walls, piers or buttresses transmitting horizontal forces to foundation.

The load-bearing capacity of a wall depends upon the spacing and effectiveness of lateral supports.

If the slenderness ratio is based on height, a horizontal lateral support (*i.e.*, floor/roof) may be deemed to be adequate if the R.C.C. floor/roof bears on wall to the extent of atleast 10 cm. In case slenderness ratio is based on effective length, a vertical support will be deemed to be adequate if cross-wall, pier or buttress extends to the extent of one-sixth of the height of the wall, has a minimum thickness of half the thickness of supported wall or 100 mm whichever is more, and is bonded to the supported wall.



(a) R.C.C. SLAB GIVING LATERAL SUPPORT TO THE WALL (b) CROSS-WALLS GIVING LATERAL SUPPORT TO THE WALL, (c) PIERS GIVING LATERAL SUPPORT TO THE WALL.

FIG. 8.3. LATERAL SUPPORT TO WALL.

National Building Code of India specifies that when the concrete slabs do not bear on a wall, as specified above, non-corrodible metal anchorages shall be provided at intervals of not more than 2 m and built into concrete slabs to a minimum distance of 40 cm. Timber floors and roofs shall be anchored by non-corrodible metal anchors having a minimum cross-section of 30 mm wide and 6 mm thick securely fastened to the joist and provided with split and upset ends or other approved means for building into the walls. The anchors shall be provided at intervals of not more than 2 m in buildings upto two storeys and 1.25 m for all storeys in other buildings.

8.4. EFFECTIVE HEIGHT OF WALL

The effective height (h) of the wall, to be used for the computation of the slenderness ratio, is the function of the actual height (H) of the wall and the conditions of lateral support. Table 8.1 gives the effective height for various conditions for supports illustrated in Fig. 8.4.

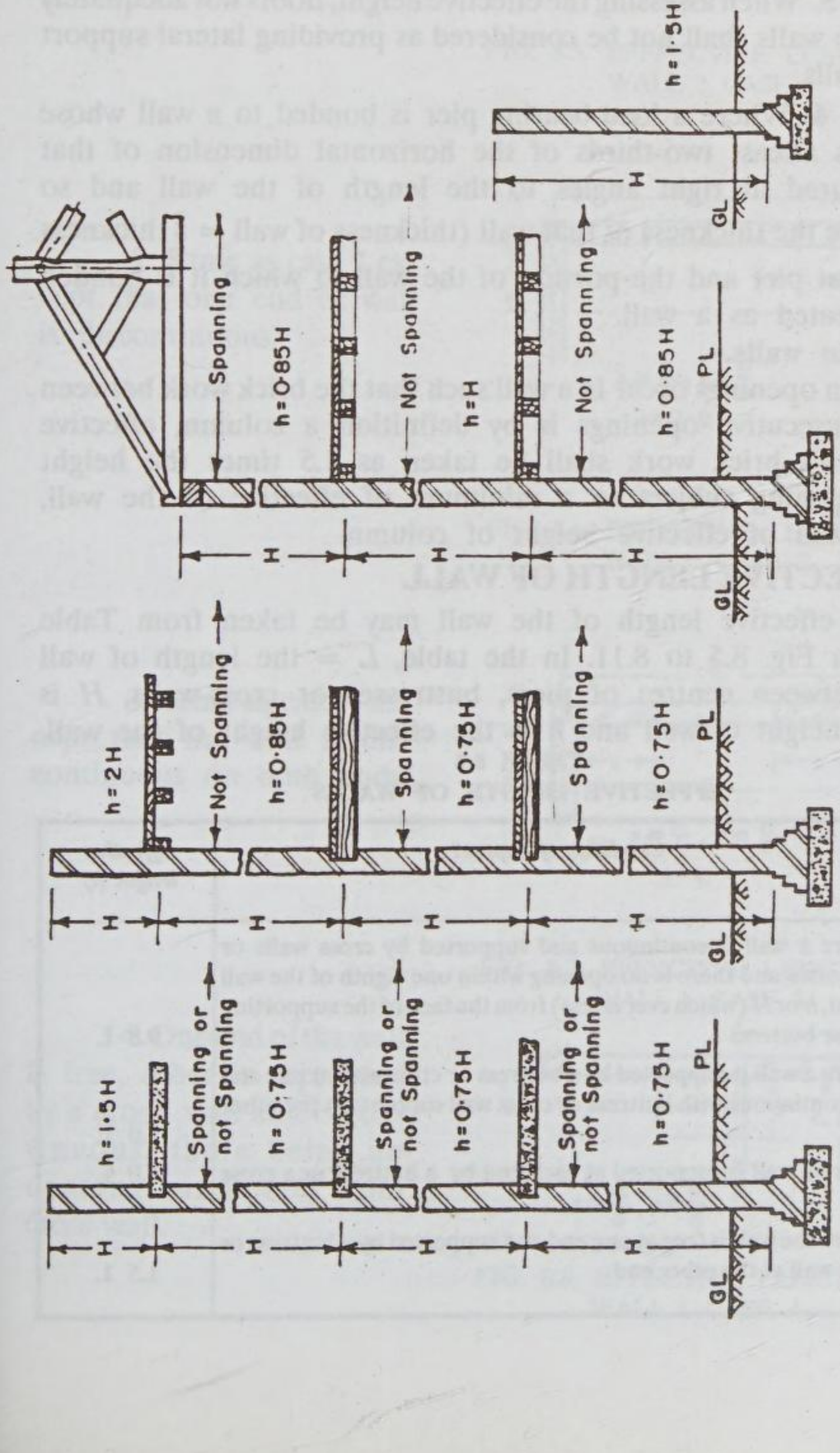
TABLE 8.1
EFFECTIVE HEIGHT OF WALL
(NATIONAL BUILDING CODE OF INDIA, SP–7 : 1970)

S.No.	*Condition of Support*	*Effective Height (H)*
1.	Adequate lateral support and partial rotational restraint at top and bottom. For example, where the floor (or roof) has a direction of span at right angles to the wall, so that the reaction to the load of the floor or roof is provided by the walls ; or where the concrete floors have a bearing on walls irrespective of the direction of span.	0.75 H
2.	Adequate lateral support and partial rotational restraint at either top or bottom and lateral restraint at other end. For example, fully braced construction which is itself adequately supported and incorporates: (*a*) timber floors immediately below or above a reinforced concrete floor, and (*b*) roof trusses above a reinforced concreted floor or the like.	0.85 H
3.	Adequate lateral support at top and bottom where the floors (or roofs) have a direction of span parallel with the wall, top and bottom, and do not bear on it, or fully braced construction which is itself adequately supported and which incorporates roof trusses and timber upper storey floors.	1.00 H
4.	Adequate lateral support and partial rotational restraint at bottom and no lateral support or rotational restraint at the top (where the wall has no lateral support at top construction not fully anchored or not fully braced).	1.50 H
5.	Free standing non load bearing members.	2.00 H

Note 1. H is the height of a wall between centres of support or the centre of support to the point near the footing, where the thickness of the wall is minimum.

Note 2. Where there is discontinuity in bond, due to damp-proof course or other materials, H should be measured from the discontinuity and the condition of end restraint at the discontinuity shall be taken as one of the lateral supports only.

Note 3. A suitable concrete element, such as a footing or



(a) RCC Floor/Roof Being on Wall Irrespective of Direction of Span.

(b) Timber Floor/Roof

(c) Timber Floor and Trussed Roof

(d) Free Standing Wall.

FIG. 8.4. EFFECTIVE HEIGHT OF WALL.

floor (irrespective of the direction of span) having bearing on or supporting a wall may be considered to provide partial restraint. In the case of roofs, the partial rotational restraint shall be assumed to be provided only when the direction of span is at right angles to the direction of wall.

Note 4. In the case of column, the effective height for both of its sides shall be considered taking into account the conditions of support at the ends.

Note 5. When assessing the effective height, floors not adequately anchored to walls shall not be considered as providing lateral support to such walls.

Note 6. Where a load-bearing pier is bonded to a wall whose thickness is atleast two-thirds of the horizontal dimension of that pier, measured at right angles to the length of the wall and so as to include the thickness of that wall (thickness of wall $= \frac{2}{3}$ thickness of pier), that pier and the portion of the wall to which it is bonded may be treated as a wall.

Openings in walls.

When openings occur in a wall such that the brick work between any two/consecutive openings is by definition a column, effective height of this brick work shall be taken as 1.5 times the height of taller opening subject to a minimum of effective of the wall, and maximum of effective height of column.

8.5. EFFECTIVE LENGTH OF WALL

The effective length of the wall may be taken from Table 8.2 or from Fig. 8.5 to 8.11. In the table, L = the length of wall from or between centres of piers, buttresses or cross-walls, H is the actual height of wall and h is the effective height of the wall.

TABLE 8.2
EFFECTIVE LENGTH OF WALLS

S. No.	*Condition of Support*	*Effective length (l)*
1.	Where a wall is continuous and supported by cross walls or buttresses and there is no opening within one eighth of the wall height, h or H (which ever is less) from the face of the supporting wall or buttress	0.8 L
2.	Where a wall is supported by a buttress or cross wall at one end and continuous with buttress or cross wall supports at the other end.	1.0 L
3.	Where a wall is supported at each end by a buttress or a cross wall	1.0 L
4.	Where the wall is free at one end and supported by a buttress or cross wall at the other end.	1.5 L

1. Wall is continuous at both ends and is supported by cross-walls of thickness $t_w/2$ or 100 mm, whichever is more; length of cross–wall is not less than $H/6$; opening in wall not closer than $H/8$ from cross-wall.

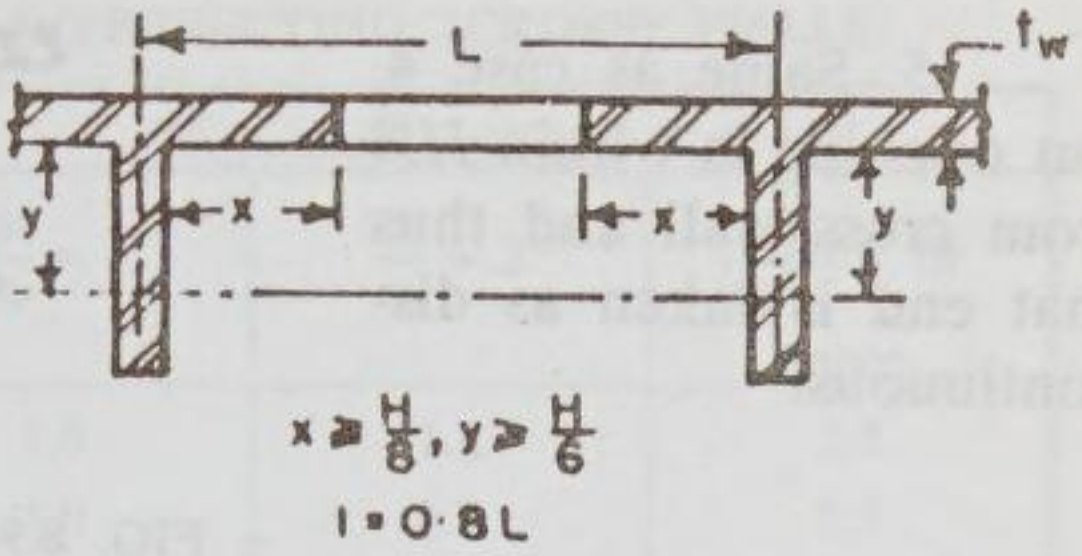


FIG. 8.5. EFFFECTIVE LENGTH OF WALL : CASE 1.

2. Same as case 1 except that one end of wall is discontinuous.

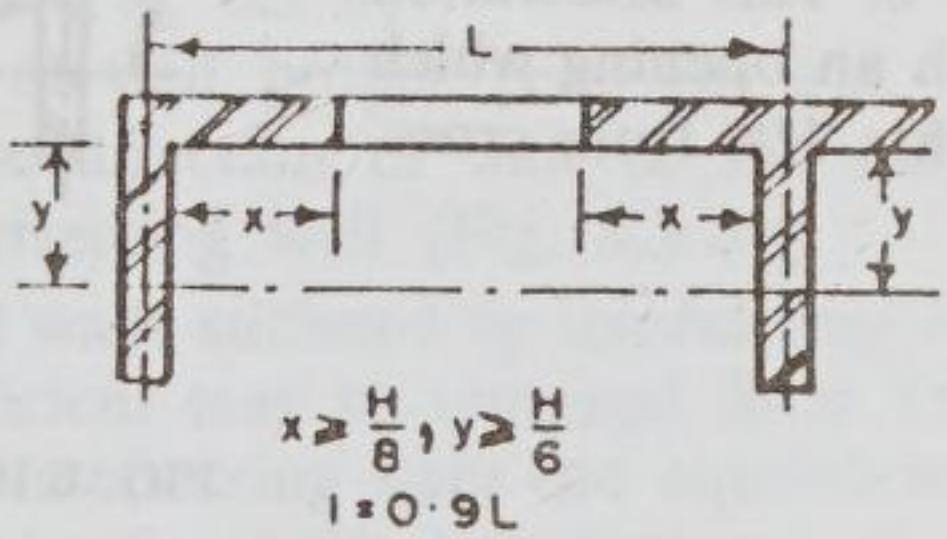


FIG. 8.6. EFFECTIVE LENGTH OF WALL : CASE 2.

3. Same as case 1 except that the wall is discontinuous on both ends.

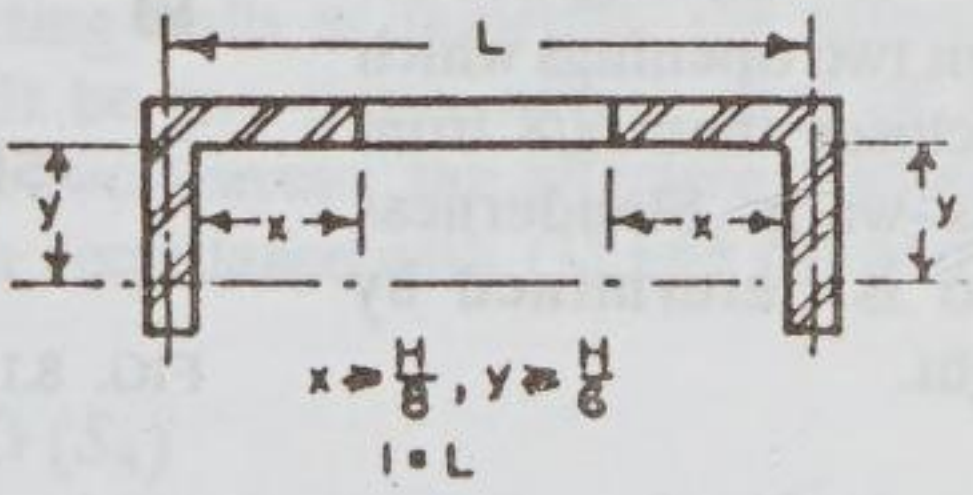


FIG. 8.7. EFFECTIVE LENGTH OF WALL : CASE 3.

4. One end of the wall is free, other is supported by a cross- wall and is continuous, there being no opening within $H/8$ from cross-wall.

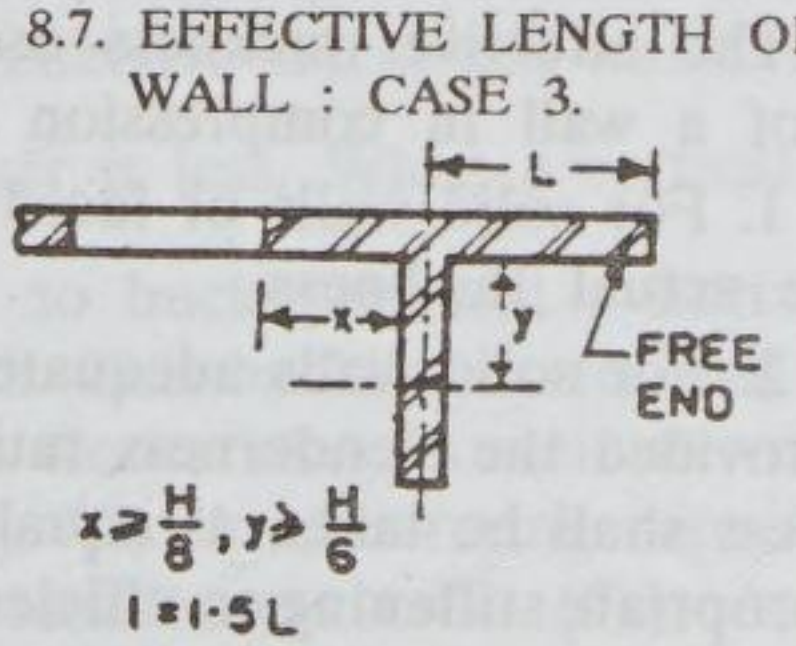


FIG. 8.8. EFFECTIVE LENGTH OF WALL : CASE 4.

5. Same as case 4, but opening is within $H/8$ from cross–wall and thus that end is taken as discontinuous.

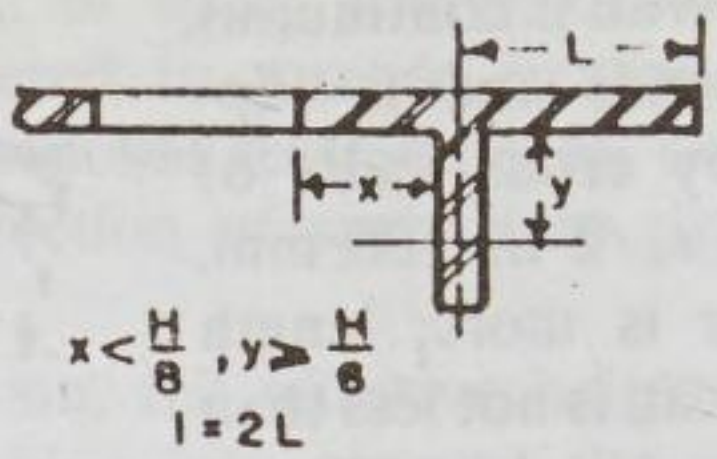


FIG. 8.9. EFFECTIVE LENGTH OF WALL : CASE 5.

6. This illustration is with an opening which is within $H/8$ from cross-wall.

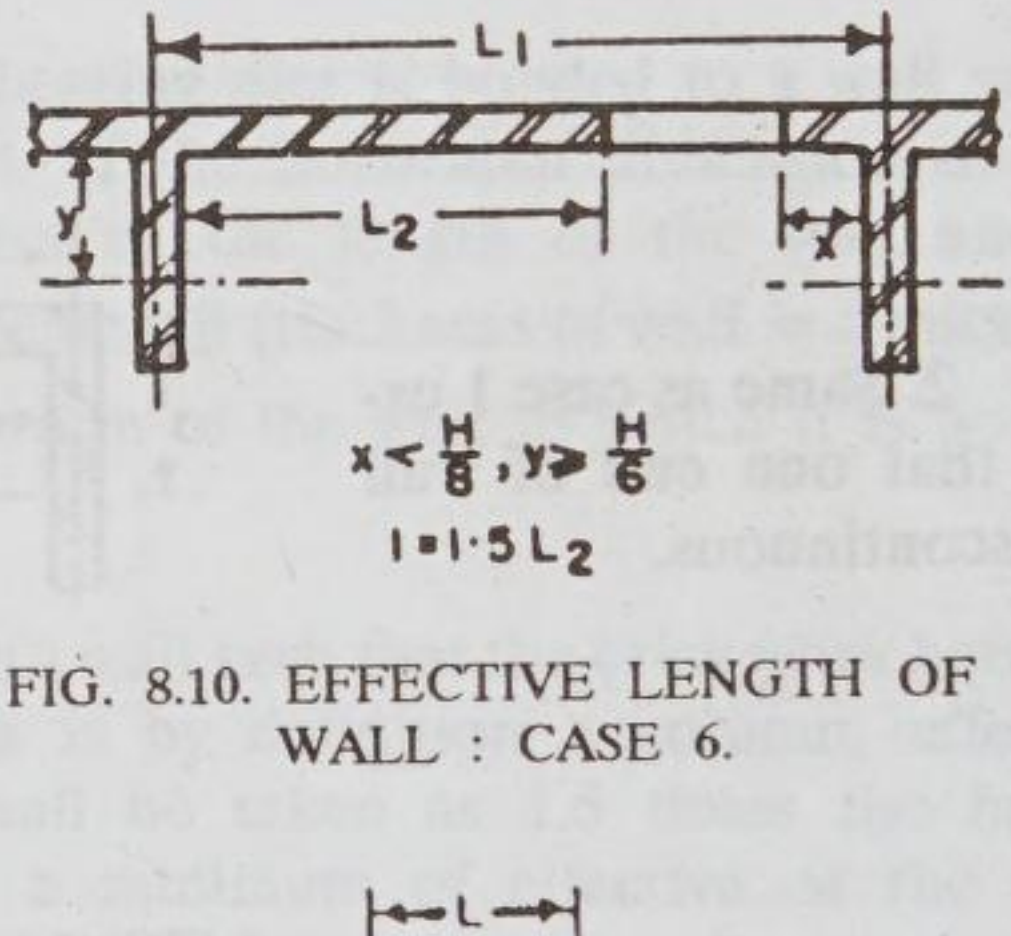


FIG. 8.10. EFFECTIVE LENGTH OF WALL : CASE 6.

7. Wall length is between two openings which are closer than $H/8$ from cross–walls. Slenderness ratio is determined by height.

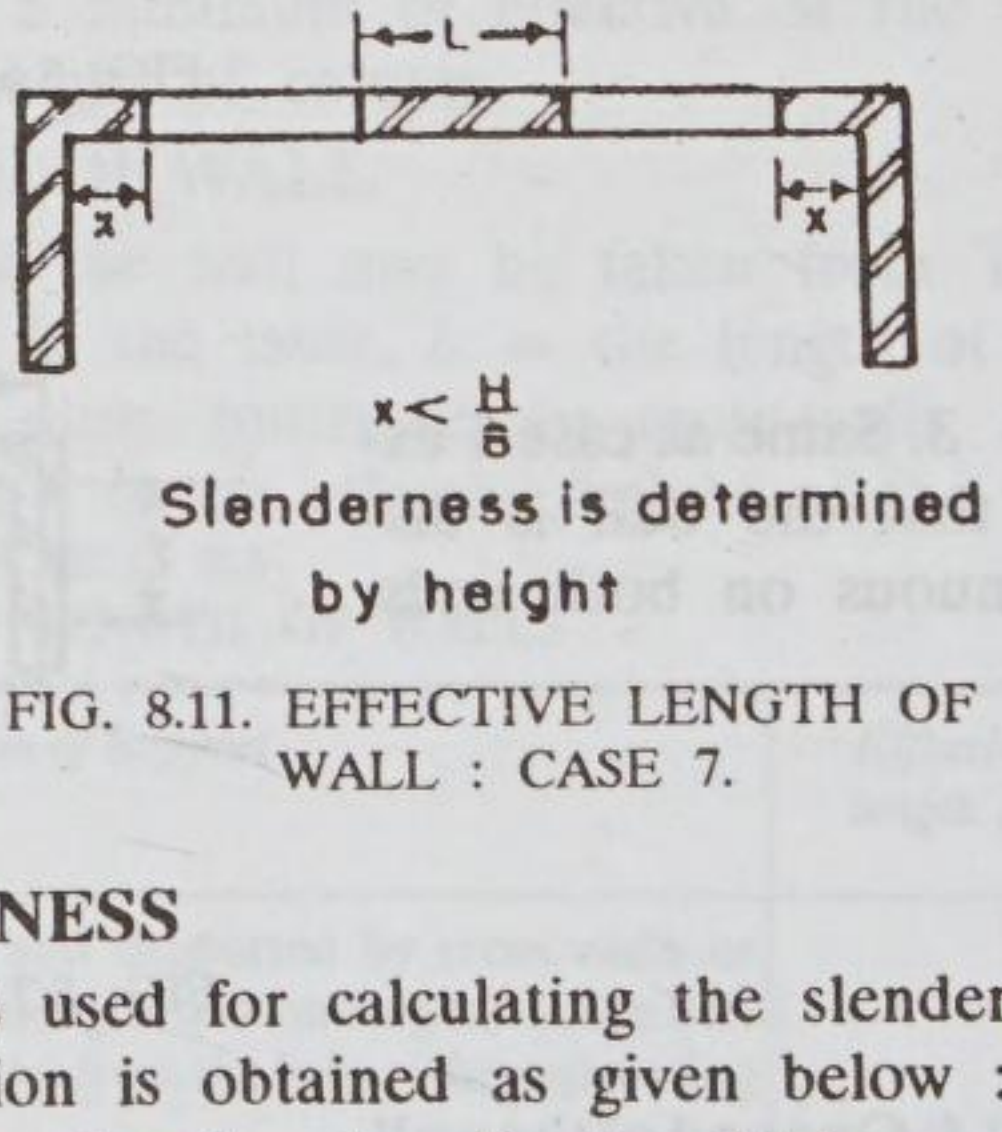


FIG. 8.11. EFFECTIVE LENGTH OF WALL : CASE 7.

8.6. EFFECTIVE THICKNESS

The effective thickness used for calculating the slenderness ratio of a wall in compression is obtained as given below :

1. For solid walls or faced walls, the effective thickness shall be the actual thickness.

2. For solid walls adequately bonded into piers or buttresses, and provided the slenderness ratio is based on height, the effective thickness shall be taken as equal to the actual thickness multiplied by appropriate stiffening co-efficient as given in Table 8.3. *No modification is, however, necessary if the slenderness ratio is based on the effective length of the wall.*

TABLE 8.3 STIFFENING CO-EFFICIENT (K_n) FOR WALLS STIFFENED BY PIERS, BUTTRESSES OR INTERSECTING (CROSS) WALLS.

S.No.	*Ratio* $\frac{s_P}{w_P}$	*Stiffening co-efficient* (K_D)		
		$\frac{t_P}{t_W} = 1$	$\frac{t_P}{t_W} = 2$	$\frac{t_P}{t_W} = 3$ or more
1.	6	1.0	1.4	2.0
2.	8	1.0	1.3	1.7
3.	10	1.0	1.2	1.4
4.	15	1.0	1.1	1.2
5.	20 or more	1.0	1.0	1.0

where s_P = centre to centre spacing of the intersecting wall.
t_P = thickness of pier (Fig. 8.3 *b*)
t_W = actual thickness of wall proper (Fig. 8.3 *b*).
w_P = pier width in the direction of wall or the actual thickness of intersecting wall (Fig. 8.3 *b*).

3. For solid walls or faced walls stiffened by intersecting wall, the appropriate stiffening co-efficient may be obtained from Table 8.3 on the assumption that the intersecting walls are equivalent to piers of width equal to the thickness of the intersecting wall and of thickness equal to three times the thickness of stiffened wall.

4. For cavity walls with both leaves of uniform thickness throughout, the effective thickness shall be two-thirds of the sum of the actual thickness of the two leaves.

5. For cavity walls with one or both leaves adequately bonded into piers, buttresses or intersecting walls at intervals, the effective thickness of the cavity wall shall be two-thirds of the sum of the effective thickness of each of the two leaves ; the effective thickness of each leaf shall be calculated in accordance with (1) and (3) above, as appropriate.

8.7. SLENDERNESS RATIO (S_R)

For a wall, the slenderness ratio shall be the effective height divided by the effective thickness and stiffening co-efficient (K_n) or the effective length divided by the effective thickness, whichever is less. Thus, $S_R = \frac{h}{t \times k_n}$ or $\frac{l}{t}$ whichever is less. When a vertical load is applied to a wall, it would tend to buckle around a horizontal axis parallel to the length of the wall. This buckling is resisted both by the horizontal supports (such as floors etc.) as well as vertical supports (*i.e.*, cross-walls, piers etc.). The load carrying capacity of wall very much depends upon its slenderness ratio (S_R). As this ratio increases, crippling stress of the wall gets reduced because of the limitations of *workmanship and elastic instability.* When S_R is

less than 30, the load-capacity of wall is a *stress problem,* while if S_R is more than 30, it becomes a *stability problem.*

From considerations of structural soundness and economy of design, most Codes control the maximum S_R so that failure is due to excessive stress rather than buckling. This *limiting value* of S_R is less for masonry built in time mortar as compared to that built in cement mortar. Similarly, limiting S_R is less for taller buildings than that for short height buildings. Table 8.4 gives the limiting values of S_R, based on *British Standard* C.P. 111–1970 (Revision November 1971) :

TABLE 8.4 LIMITING VALUES OF SLENDERNESS RATIO

No. of storeys	*Thickness of wall*	*Maximum Values of S_R*	
		Mortar containing cement	*Mortar not containing cement*
Not exceeding 2	Any	27	20
Exceeding 2	(*i*) Walls of 90 mm thick-ness or more	27	13
	(*ii*) Wall less than 90 mm thickness	20	13

8.8. BASIC COMPRESSIVE STRESS (f_b)

The basic compressive stress (f_b) of masonry depends upon the crushing strength of masonry units (brick) and type of mortar used. Table 8.5 gives the basic compressive stresses of masonry. It should be noted that basic compressive stresses, given in Table 8.5 also depends upon surface characteristics, water absorption property of units, uniformity of shape and size of units, thickness of joints etc. Thus, strictly speaking, the basic stresses given in Table 8.5 would not hold good for concrete blocks, sand lime bricks and stone. In absence of any other specific provisions in the Code, the basic stresses given in Table 8.5 may be adopted for ashlar stone and coursed stone masonry with co-efficients of 1.25 and 0.75 respectively. This is so because ashlar masonry, requiring thin mortar joints would give high basic stress while coursed stone masonry with irregular stones and thicker joints would give lesser values of basic stresses.

8.9 STRUCTURAL DESIGN OF WALLS

British Code C.P. 111 divides the walls into two categories : *non-calculated walls* and *calculated walls. Non-calculated walls* are those which are used as thin panel walls in framed structures and which do not carry roof load. The design of such walls is based on certain *rules* and not on the basis of calculations. On the other hand, *calculated walls* are the load-bearing walls of high rise buildings,

which support floors and roof, and the design of which is based on *calculations* and not on rules. The method of design of such walls is commonly known as *calculated masonry method*. The *calculated masonry method* can be applied in *two ways:*

1. Design by use of nomograms.
2. Design by structural analysis.

DESIGN BY USE OF NOMOGRAMS

Results of structural analysis, based on *'calculated masonry method'*, for certain loadings and span, have been presented in the form of nomograms, by the National Building Code of India (SP : 7–1920). Though the nomograms presented in the Code cover buildings upto 6 storeys, it is viewed that there is risk of some portion of the structure getting overstressed, on account of unfavourable location and shape of openings and occurrence of concentrated loads thus endangering the structural safety of tall buildings. It is desirable, therefore, to use the nomograms to design buildings upto 3 storey height. Buildings exceeding 3 storeys in height should be designed by detailed *structural analysis.*

Figs. 8.12 and 8.13 give two typical nomograms for the design of walls of residential buildings, for class 200 loading with 2.8 m and 3.2 m storey heights respectively. For other cases of loadings and storey heights, reference may be made to the National Building Code of India (SP : 7–1920).

The nomograms for determining thickness of brick wall contain nine vertical lines. From left to right, the vertical lines represent as follows :

Line 1 : Basic stress
Line 2 : Storeys
Line 3 : Reference line 1
Line 4 : Span point
Line 5 : Reference line 2
Line 6 : Percentage of opening
Line 7 : Thickness of wall for span of 3.0 m
Line 8 : Thickness of wall for span of 3.6 m
Line 9 : Thickness of wall for span of 4.2 m.

Line 1 : Basic stress. Basic stress for the type of masonry is taken from Table 8.5.

Line 2 : Storeys.Though the nomograms can be used for buildings upto 6 storeys height, it is preferable to use it only upto 3 storey height. For use of nomograms in the case of multi-storeyed buildings, the wall thickness at each floor is found by passing the line through the *number* of storeys above that section.

Line 3 : Reference line 1. This line fixes a point on the line for any combination of basic stress and storeys.

Line 4 : Span point. The fourth line has a span point through

which all lines shall pass through for arriving at the wall thickness.

Line 5 : Reference line 2. This reference line also fixes a point on the line for any combination of values for basic stress and storeys.

Line 6 : Percentage of openings. This line takes care of openings provided on the wall, for windows, doors, ventilators etc. For example, if a wall, 6 m long, has a door of 1 m width and window of 1.5 m width, the gross horizontal area of wall, at a plane where there is no opening, will be $6\,t_w$ m^2, where t_w is the thickness of wall in metres. The area of opening at the window level will be $= (1 + 1.5)\,t_w = 2.5\,t_w$ m^2. Hence the percentage opening $= \frac{2.5\,t_w}{6\,t_w} \times 100 \approx 41.7\%$. Nomograms are valid upto 50% openings.

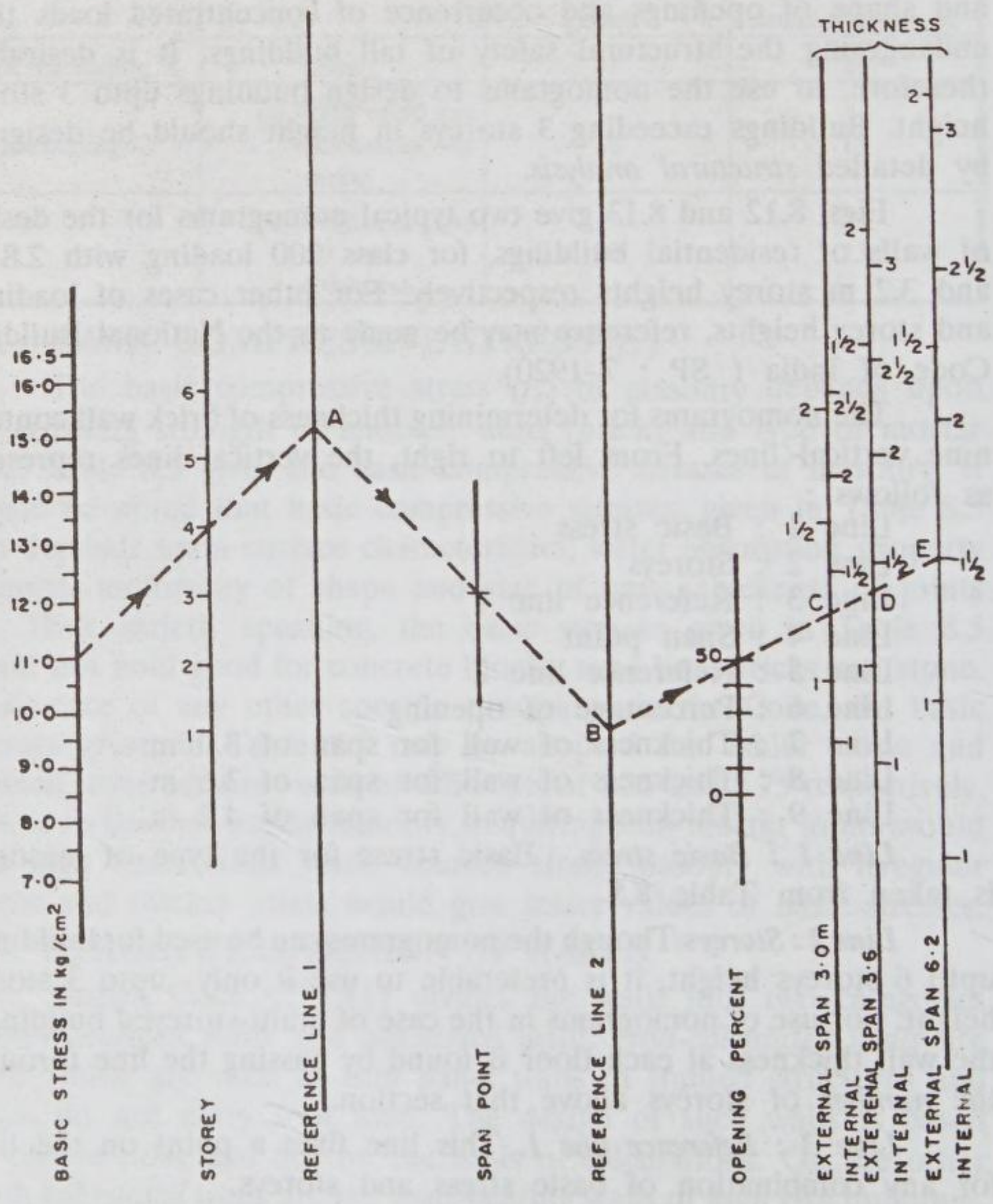


FIG. 8.12. NOMOGRAM FOR WALL THICKNESS :
RESIDENTIAL BUILDING: CLASS 200 LOADING,(LIVE LOAD)
DEAD LOAD : 415 kg/m^2 (4.15 kN/m^2) STOREY HEIGHT : 2·8 m

Lines 7, 8, 9 : Thickness of wall. The last three lines give the thickness of wall for three spans of the rooms (*i.e.* 3.0 m, 3.6 m and 4.2 m). Thickness are indicated on both the sides of these lines. The bold markings on the left hand side of the lines give the thickness of the external walls and the dotted markings on the right side of the lines give the thickness for internal walls. Internal walls are analyzed as walls having spans on either side. The numbers 1, $1\frac{1}{2}$, 2 etc. on these lines indicate the (number of) brick thickness.

Method of use of nomograms

The method of use of nomogram for determining the thickness of brick wall has been illustrated in Figs. 8.12 and 8.13 by a dotted line. The following procedure may be adopted :

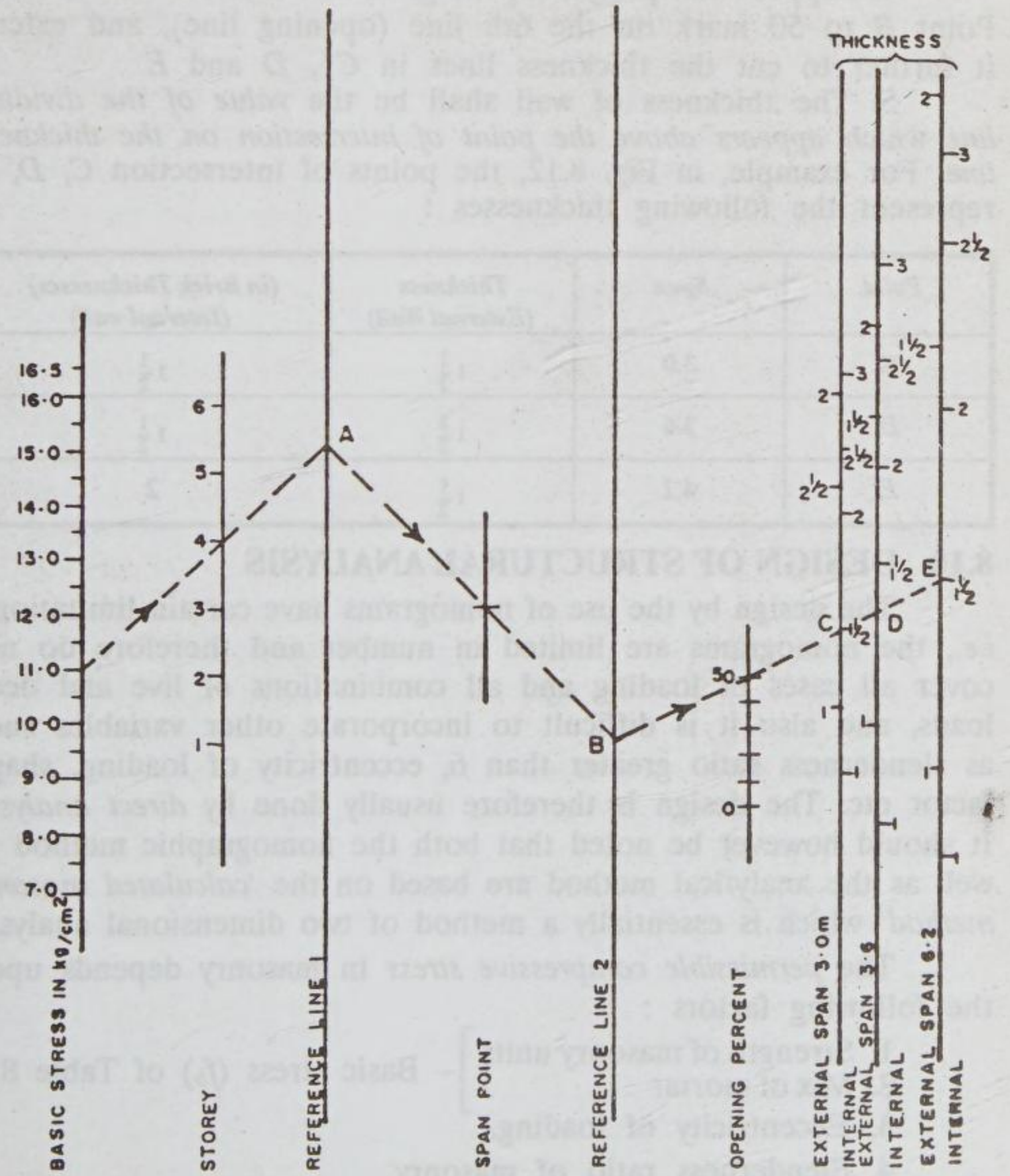


FIG. 8.13. NOMOGRAM FOR WALL THICKNESS
RESIDENTIAL BUILDING : CLASS 200 LOADING.
DEAD LOAD : 415 kg/m^2 (4.15 kN/m^2) STOREY HEIGHT : 3.2 m

1. Choose the basic stress corresponding to the properties of masonry units and type of mortar to be used. For example, if we use bricks having crushing strength of 140 kg/cm^2 (14 N/mm^2) and 1 : 1 : 6 cement-lime mortar, the basic compressive stress, found from Table 8.5, will be 11 kg/cm^2 (1.1 N/mm^2). This stress of 11 kg/cm^2 is marked on the first line of the nomograms (Fig. 8.12.).

2. If the building has 4 storeys, and the wall is to be designed at the ground level, the point of 11 kg/cm^2 is joined to storey 4 of the second line (story line), and extended to cut reference line No. 1 at point *A*.

3. Join *A* to the span point and prolong further to cut the reference line No. 2 in point *B*.

4. Suppose the percent openings in the wall are 50. Join the Point *B* to 50 mark on the 6th line (opening line), and extend it further to cut the thickness lines in *C* , *D* and *E*.

5. The thickness of wall shall be the *value of the dividing line which appears above the point of intersection on the thickness line.* For example, in Fig. 8.12, the points of intersection *C, D, E* represent the following thicknesses :

Point	*Span*	*Thickness (External Wall)*	*(in Brick Thicknesses) (Internal wall)*
C	3.0	$1\frac{1}{2}$	$1\frac{1}{2}$
D	3.6	$1\frac{1}{2}$	$1\frac{1}{2}$
E	4.2	$1\frac{1}{2}$	2

8.10. DESIGN OF STRUCTURAL ANALYSIS

The design by the use of nomograms have certain limitations, *i.e.*, the nomograms are limited in number and therefore do not cover all cases of loading and all combinations of live and dead loads, and also it is difficult to incorporate other variables such as slenderness ratio greater than 6, eccentricity of loading, shape factor etc. The design is therefore usually done by *direct analysis.* It should however be noted that both the nomographic method as well as the analytical method are based on the *'calculated masonry method'* which is essentially a method of two dimensional analysis.

The *permissible compressive stress* in masonry depends upon the following factors :

1. Strength of masonry units
2. Mix of mortar

] – Basic stress (f_b) of Table 8.5.

3. Eccentricity of loading.
4. Slenderness ratio of masonry.
5. Shape factor.

TABLE 8.5 BASIC COMPRESSIVE

S. No.	Description of Mortar (With Designation)	Mix (Parts by Volume)			
		Cement	Lime	Lime-Pozzolana	Pozzolan
(1)	(2)	(3)	(4)	(5)	(6)
1.	Cement (H_1)	1	0 – $\frac{1}{4}$ C*	–	–
2.	Cement (H_2)	1	$\frac{1}{2}$ C*	–	–
3.	Cement-lime (M_1)	1	1 C	–	–
4.	Cement-lime (M_2)	1	2 B	–	–
5.	Cement (M_2)	1	–	–	–
6.	Lime-pozzolana mixtured (M_2)	–	–	1	–
7.	Cement-lime (L_1)	1	3 B or C	–	–
8.	Hydraulic lime (L_2)	–	1 A	–	–
9.	Lime pozzolana (L_2)	–	1 C	–	1
10.	Lime (L_2)	–	1 B	–	–

* Note 1. *A*, *B*, *C* are the types of Lime conforming to relevant Indian Standards.

* Note 2. Inclusion of Lime in cement mortars is optional.

Note 3. The above Table is valid for slenderness ratio upto 6.

Note 4. Symbols *H*, *M* and *L* denote high strength, medium strength and low st

(10 $kg/cm^2 \approx 1\ N/mm^2$)

ES (f_b) FOR MASONRY MEMBERS

nd	Hardening time after completion of work	Basic stress in kg/cm² corresponding to masonry units with crushing strength (kg/cm²)								
		35	70	105	140	175	210	280	350	440
)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)
	7	3.5	7.0	10.5	12.5	14.5	16.5	21.0	25.0	30.5
	14	3.5	7.0	10.0	11.5	13.0	14.5	17.5	21.0	25.0
	14	3.5	7.0	10.0	11.0	12.0	13.0	16.0	19.0	22.0
	14	3.5	5.5	8.5	10.0	11.0	12.0	14.5	16.5	19.0
	14	2.5	5.0	7.0	8.0	9.0	10.0	12.0	14.0	16.0
	14	2.5	5.0	7.0	8.0	9.0	10.0	12.0	14.0	16.0
	28	2.5	4.0	5.5	6.0	6.5	7.0	7.5	8.5	9.5

(To face Page 336)

tars.

Basic stresses (f_b). The basic stresses are given in Table 8.5, which are normally applicable for bricks. For ashlar and coursed rubble masonry, factors of 1.25 and 0.75 respectively may be used, as stated earlier.

Area Reduction factor. The Code has given in an expression of the reduction factor (K_a) due to small area of wall, which is based on the consideration that there is statically greater chance of failure of a small section, from weak individual units or variability of workmanship. The National Building Code recommends that where the cross-sectional area of the wall does not exceed 3000 cm^2, the basic stress (f_b) should be multiplied by a reduction factor given by

$$K_a = 0.75 + \frac{A}{12000} \qquad ...(8.1)$$

where A = Area (in cm^2) of the horizontal cross section of the wall.

Eccentricity of Loading

When the floor slabs (or roof slabs) transferring load to the wall do not bear on it for the full width of the wall, the vertical load transferred to will act eccentrically. Eccentric vertical loads or axial loads plus lateral loads on wall cause bending stresses in addition to axial stresses.

Fig. 8.14 shows a wall of thickness t, carrying axial load W_1 and eccentric load W_2 at eccentricity e, per unit length of wall. The resultant load W will have an eccentricity $\bar{e}$ which can be found by taking moments around the centroid of the section AB:

$$W \cdot \bar{e} = W_1 \times 0 + W_2 \times e$$

(Where $W = W_1 + W_2$)

$$\therefore \qquad \bar{e} = \frac{W_2 \cdot e}{W_1 + W_2} \qquad ...(8.3)$$

Code designates $\bar{e}$ as the *equivalent eccentricity.*

The eccentric load will cause both *axial stress* ($= W/t$), and *bending stress*

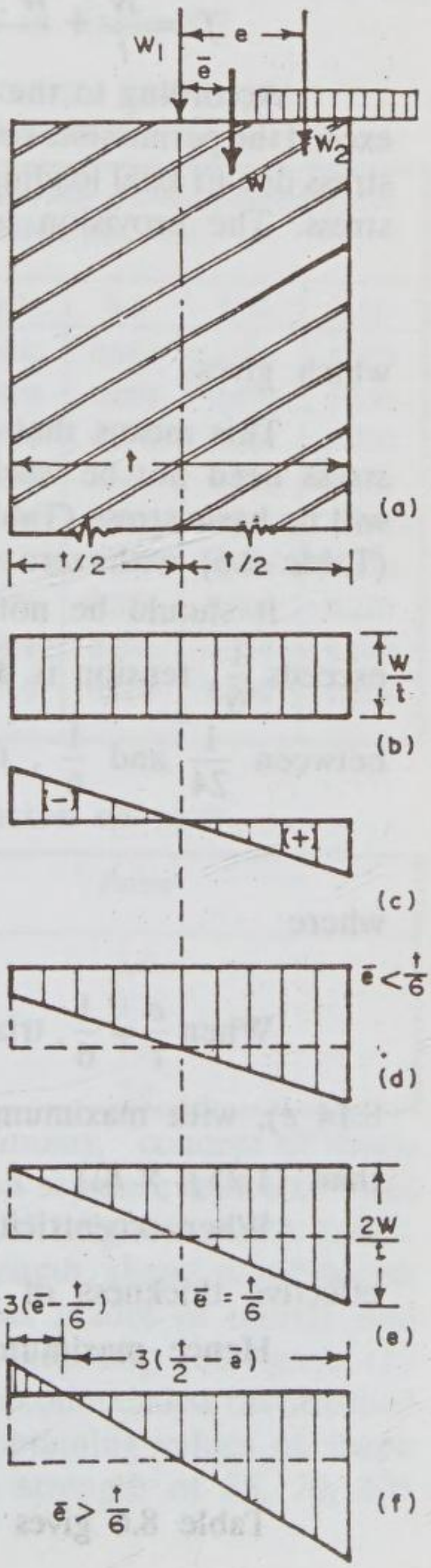


FIG. 8.14.

$\left(= \frac{W.\bar{e} \times 6}{t^2} \right)$. Thus, the stress f at extreme fibres is given by

$$f = \frac{W}{t} \pm \frac{6\,W\bar{e}}{t^2}$$

or
$$f = \frac{W}{t}\left(1 \pm \frac{6\,\bar{e}}{t}\right) \qquad ...(8.3)$$

The total compressive stress f_c at the extreme fibres of section AB is

$$f_c = \frac{W}{t} + \frac{W.\bar{e} \times 6}{t} = \frac{W}{t}\left(1 + \frac{6\,\bar{e}}{t}\right) \qquad ...(8.3a)$$

According to the code, additional stress due to bending could exceed the permissible compressive stress (axial) by 25 percent, provided stress due to axial loading does not exceed the permissible compressive stress. The provision leads us to :

$$\frac{W}{t} \times \frac{6\,\bar{e}}{t} \leq \frac{1}{4}\,\frac{W}{t}$$

which gives
$$\frac{\bar{e}}{t} \leq \frac{1}{24} \qquad ...(8.4)$$

This means that up to an *eccentricity ratio* of 1/24, bending stress need not be worked out. The permissible compressive stress will be basic stress (Table 8.5) multiplied by appropriate stress factor (Table 8.6) with zero eccentricity ratio.

It should be noted (Eq. 8.3 *a*) that when eccentricity ratio exceeds $\frac{1}{6}$, tension is developed on one side. For eccentricity ratio between $\frac{1}{24}$ and $\frac{1}{6}$, the stress f_c in the member is

$$f_c = \left(\frac{W}{t} + \frac{W\bar{e} \times 6}{t^2}\right) \leq 1.25\,f_b \times K_s \qquad ...(8.5)$$

where
f_b = Basic stress (Table 8.5)
K_s = Stress factor (Table 8.6)

When $\frac{e}{t} = \frac{1}{6}$, triangular stress distribution will take place (Fig. 8.14 *e*), with maximum stress equal to $\frac{2\,W}{t}$, which should be less than $(1.25\,f_b \times K_s)$

When eccentricity ratio exceeds 1/6, tension is developed, and effective thickness of masonry will be $3\left(\frac{t}{2} - \bar{e}\right)$.

Hence maximum stress in masonry will be

$$f_c = \frac{2\,W}{3\left(\frac{t}{2} - \bar{e}\right)} = 1.25\,f_b \times K_s \qquad ...(8.6)$$

Table 8.6 gives the stress factors (K_s) for slenderness ratio

and eccentricity of loading.

Shape Factor

The shape factor takes into account the effect of the shape of the masonry unit *(i.e.*., brick). The basic stresses (f_b) of Table 8.5 are suitable when the units are of common brick shape, but may be unnecessarily low for some units whose ratio of height to thickness is greater than that of the common brick. For units of crushing strength not greater than 55 kg/cm^{2}and with ratio of height to thickness as laid greater than 0.75 but not greater than 3, the basic stresses (f_b) may be modified by the factor specified in Table 8.7.

TABLE 8.6
STRESS FACTORS (K_s) FOR SLENDERNESS RATIO AND ECCENTRICITY OF LOADING

S.No.	*Slenderness Ratio*	*Stress factors for equivalent eccentricity of loading divided by the thickness of members (i.e., $\bar{e}$ /t)*						
		0	0.04	0.1	0.2	0.3	0.33	0.50
1.	6	1.000	1.000	1.000	0.996	0.984	0.980	0.970
2.	8	0.920	0.920	0.920	0.910	0.880	0.870	0.850
3.	10	0.840	0.835	0.830	0.810	0.770	0.760	0.730
4.	12	0.760	0.750	0.740	0.706	0.664	0.650	0.600
5.	14	0.670	0.660	0.640	0.604	0.556	0.540	0.480
6.	16	0.580	0.565	0.545	0.500	0.440	0.420	0.350
7.	18	0.500	0.480	0.450	0.396	0.324	0.300	0.230
8.	21	0.470	0.448	0.420	0.354	0.276	0.250	0.170
9.	24	0.440	0.415	0.380	0.310	0.220	0.190	0.110

TABLE 8.7
MODIFICATION FACTORS FOR SHAPE OF UNIT

Ratio of Height to width of brick or block	*Factor*
0.75	1.0
1.0	1.2
1.5	1.6
2.0 to 3.0	2.0

In the *Draft British Standard for Masonry,* concept of shape in modification factor has been changed and separate tables of basic stress have been given for varying values of height-to-width ratios. It is seen that for units of a particular strength, shape modification factor is more or less the same for various grades of mortar and this factor is very close to unity for units exceeding 150 kg/cm^2 (15 kN/mm^2) crushing strength. It is therefore recommended that instead of Table 8.7, Table 8.8 may be used for obtaining values of shape modification factors for units of crushing strength of 55, 70, 105 and 140 kg/cm^2.

TABLE 8.8
REVISED SHAPE MODIFICATION FACTORS

Height to width ratio of units	*Shape modification factors for units of crushing strength (kg/cm²)*			
	55	70	105	140
upto 0.75	1.0	1.0	1.0	1.0
1.0	1.2	1.1	1.1	1.0
1.5	1.5	1.3	1.2	1.1
2.0 to 4.0	1.8	1.5	1.3	1.2

Note : 10 kg cm $\approx$ 1 N/mm²

From Table 8.8, it is concluded that for units of strength greater than 140 kg/cm², effect of shape on stress is negligible and could be ignored.

Tensile Stress in masonry

For mortar not weaker than a 1 : 1 : 6 cement : lime : sand mix or equivalent, the permissible tensile stress in bending shall not exceed 1 kg/cm². For weaker mortars, the designer should assume that, that part or section (upto which tensile stress is caused) will be inactive and the remainder will carry the compressive stress, as expressed in Fig. 8.6.

Permissible shear stress.

In case of walls built in mortar not weaker than 1 : 1 : 6 cement : lime : sand mix and resisting horizontal forces in the plane of the wall, the permissible shear stress, calculated on the area of the horizontal mortar bed joint, shall be taken as 1.5 kg/cm².

8.11. CONCENTRATED LOAD

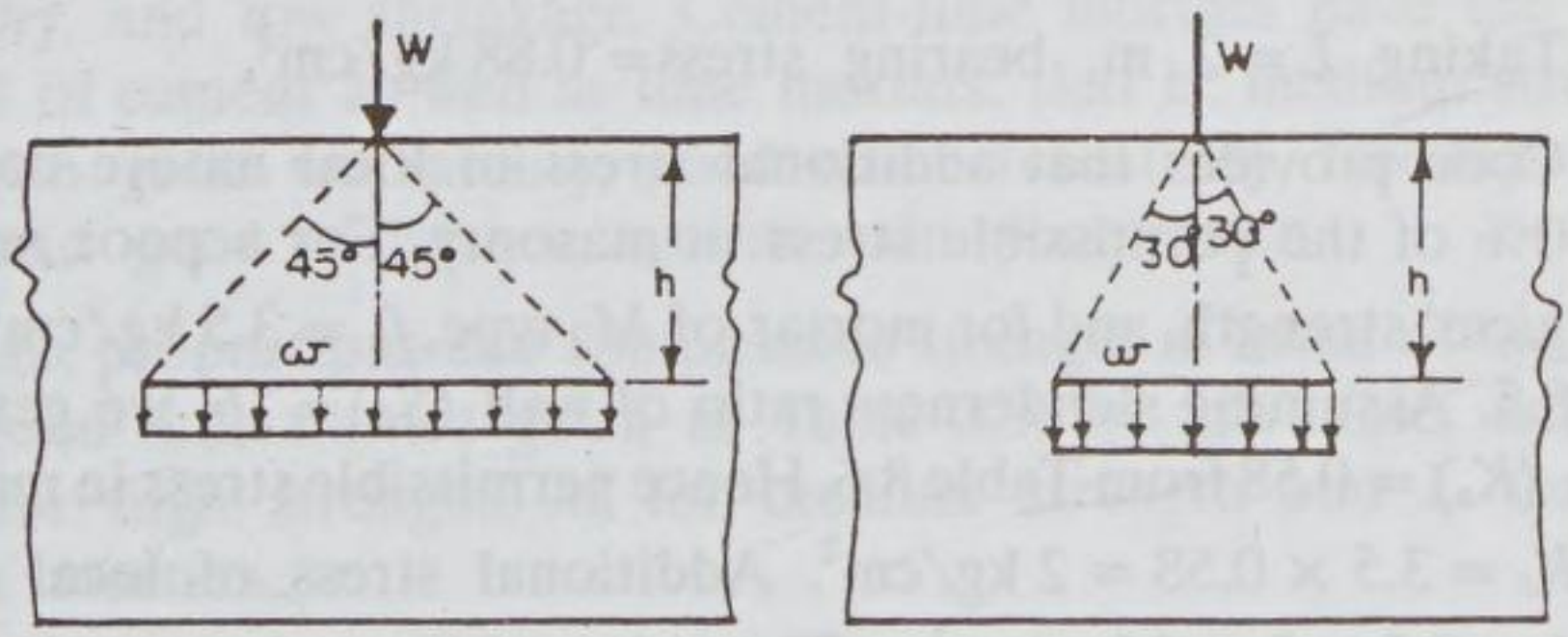
A wall has to support concentrated loads of girders, beams etc. supported on it. If the bearing area under a load does not exceed $\frac{1}{4}$ of the total cross-sectional area of the member supporting the load, it may be termed as concentrated load. The above definition given in the commentry of the *Australian Code* does not mention as to how to deal with a case when bearing is more than $\frac{1}{4}$ but less than 1. The *City of New York Building Code* gives some guidance in this regard and stipulates that allowable compressive stress provided the area of bearing does not exceed $\frac{1}{3}$ of the area of member supporting the load and the least distance between the edges of the loaded and unloaded area is a minimum of $\frac{1}{4}$ of the parallel side dimension of the loaded area. For bearing on the full area, the allowable bearing stress shall be taken as equal to the allowable compressive stress.

For reasonably concentric bearing areas greater than $\frac{1}{3}$ but less than full area, the allowable bearing stress shall be interpolated between 1.5 and 1.0 times the allowable compressive stress.

If the bearing stress in masonry wall is not within permissible limits, concrete bed block should be introduced below the load to bring down the stress in masonry to safe limit. Width of the bed block is kept the same as thickness of wall and length is calculated such that stress in wall equals permissible stress. Maximum effective length of a bed block can be either centre to centre distance of concentrated loads or $b + 4\,t_w$, whichever is less. Depth of bed block should be equal to its lengthwise projections from the bearing of load, that is, assuming 45° angle of dispersal of load within the bed block.

Dispersion of Concentrated Load

National Building Code of India (1970) puts no limit on the extent of dispersal of a concentrated load. However, it recommends that angle of dispersion shall not be taken greater than 45°, as shown in Fig. 8.15 (*a*). The *City of New York Building Code* stipulates that the length of a wall to be considered effective in resisting a concentrated load shall not exceed the centre to centre distance between the loads, nor shall it exceed the width of the bearing plus four times the wall thickness. It is further clarified by *Amrhein* that even when a bearing plate is provided under the concentrated load, maximum length of wall for distribution of concentrated load cannot exceed bearing plus 4 times the thickness of wall.



(a) 45° Dispersion (b) 30° Dispersion

FIG. 8.15. DISPERSION OF CONCENTRATED LOAD IN MASONRY.

Brick Institute of America recommends dispersal angle of 30° based on results of research in USA which is in agreement with German and Swiss thinking on this subject. For our Code, dispersal angle of 30° (Fig. 8.15 *b*) would be more appropriate since strength of bricks in India is rather low, resulting in less arching and greater load on lintels.

8.12. STRESS CONCENTRATION BELOW LINTEL BEARINGS

Concentrated stress of local nature occurs under a lintel support. Fig. 8.16 shows a lintel of span L metres, having bearing of $\frac{L}{10}$ m on each side. The wall thickness is t_w cm. Assuming an angle of spread of 30°, the lintel will support a load of equilateral triangle of side L.

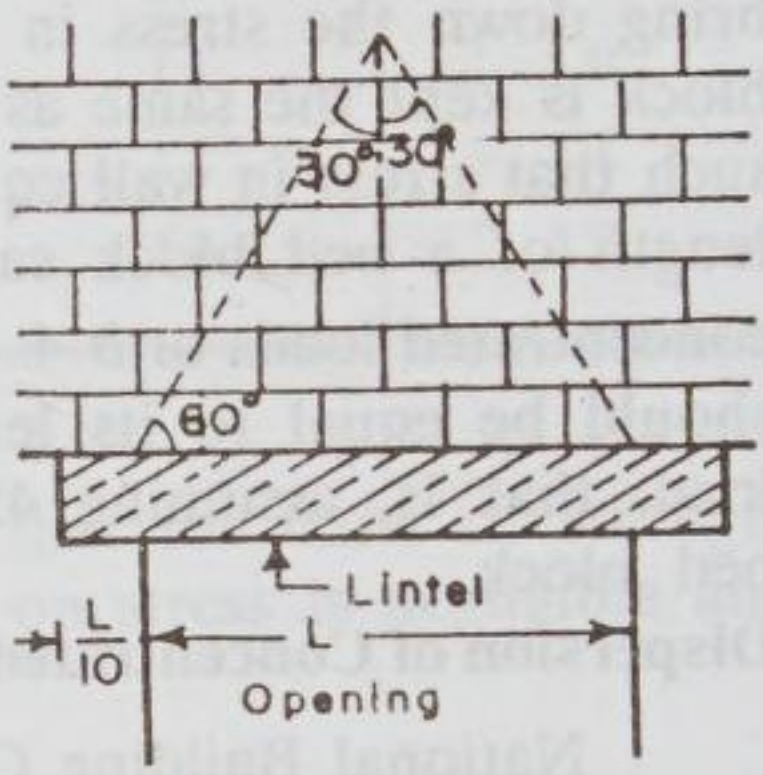


FIG. 8.16.

$$\text{Total load} = 0.44\,L^2 \times \frac{t_w}{100} \times W_u$$

where W_u = unit weight of masonry,

Taking $W_u = 2000$ kg/m^3 (20 kN/m^3).

$$W = 0.44\ L^2 \times \frac{t_w}{100} \times 2000$$

$$\text{Bearing area} = 2 \times \frac{L}{10} \times t_w \text{ cm}^2$$

$$\therefore \text{ Bearing stress on supports} = \frac{0.44\,L^2 \times \frac{t_w}{100} \times 2000}{2 \times \frac{L}{10} \times 100 \times t}$$

$$= 0.44\,L \text{ kg/cm}^2$$

Taking $L = 2$ m, bearing stress $= 0.88$ kg/cm^2.

Code provides that additional stress of local nature could be upto 50% of the permissible stress in masonry. For a poor masonry of 35 kg/cm^2 strength, and for mortar of M_2 type, $f_b = 3.5$ kg/cm^2, from Table 8.5. Assuming slenderness ratio of wall $(S_R) = 16$, we get stress factor $(K_s) = 0.58$ from Table 8.6. Hence permissible stress in masonry, $f = f_b \,.\, K_s = 3.5 \times 0.58 = 2$ kg/cm^2. Additional stress of local nature could be $0.5 \times 2 = 1$ kg/cm^2. Thus the additional stress of local nature (*i.e.*, 0.88 kg/cm^2) is well within 50 percent of permissible stress even in case of masonry units of very low strength.

For masonry units of moderate strength of 70 kg/cm^2, we get $f_b = 5.5$ kg/cm^2 from Table 8.5, for M 2 type mortar. Local stress for this masonry can go upto $\frac{5.5 \times 0.58}{2} = 1.6$ kg/cm^2

Thus $0.44\,L = 1.6$

or $$L = \frac{1.6}{0.44} = 3.5 \text{ m.}$$

Thus in all practical cases, additional stress of local nature under lintel supports remains within safe limits.

8.13. MORTAR SELECTION

Requirements of a good mortar for masonry are : strength, workability, water retentivity and low drying shrinkage. A strong mortar will have adequate crushing strength as well as adequate tensile and shear strength. It is necessary that mortar should attain initial set early enough to enable work to proceed at a reasonable pace. At the same time it should gain strength within reasonable period so that masonry is in a position to take load early. A workable mortar will hang from the trowel and will spread easily. A mortar with good water retentivity will not readily loose water and stiffen on coming in contact with masonry units, and will remain plastic long enough to be easily adjusted in line and level. This property of good water retentivity will enable the mortar to develop good bond with masonry units, so that masonry has adequate resistance against rain penetration.

Mortars could be broadly classified as *cement mortars, lime mortars and cement-lime mortars.* Cement mortars set early and gain strength quickly. Rich cement mortars, though having good strength have high shrinkage and are more liable to cracking. Lime mortars gain strength slowly and have low ultimate strength. The main advantage of lime mortar lies in its good workability, good water retentivity, and low shrinkage. Cement-lime mortars have the good qualities of cement as well as lime mortars, that is, medium strength along with good workability, good water retentivity, freedom from cracks and good resistance against rain penetration.

Mix proportions and compressive strength of some of the commonly used mortars are given in Table 8.9. In this table, letter *H* stands for high strength, *M* for medium strength and *L* for low strength mortars.

It has been stated earlier that mortar strength shall in general not be greater than that of the masonry unit. A unnecessarily strong mortar concentrates the effect of any differential movement of masonry in fewer and wider cracks while a weak mortar (mortar having more of lime and less of cement will accommodate movements and cracking will be distributed as thin hair cracks which are less noticeable. Also, stresses due to expansion of masonry units are reduced, if a weak mortar is used.

TABLE 8.9
MIX PROPORTION AND STRENGTH OF COMMONLY USED MORTARS FOR MASONRY

S.No.	*Mix*			*Minimum compressive strength*		*Mortar Type*
	Cement	Lime	Sand	Kg/cm^2	N/mm^2	
1.	1	$0-\frac{1}{4}C$	3	100	10	H 1
2 (*a*).	1	0	4	75	7.5	H 2
2 (*b*).	1	$\frac{1}{2}C$	$4\frac{1}{2}$	60	6	
3 (*a*).	1	0	5	50	5	M 1
3 (*b*).	1	1C	6	30	3	
4 (*a*).	1	0	6	30	3	M 1
4 (*b*).	1	2C	9	20	2	
4 (*c*).	0	1A	2–3	20	2	
5 (*a*).	1	0	8	7	0.7	L 1
5 (*b*).	1	3C	12	7	0.7	
6.	0	1 B or C	2–3	5	0.5	L 2

Note : A,B,C denote eminently hydraulic lime, semi-hydraulic lime and fat lime respectively, as stipulated in Indian Standards.

It is preferable to use composite mortars of cement, lime and sand, in appropriate proportions. Fig. 8.17, based on *BSR* Digest 61 (*second series*), illustrates the relation between strength of mortar and brick-work for a number of mortar mixes when bricks of medium strength are used. As the proportion of lime in mortar is increased, though mortar looses strength, reduction in strength of brick work is not much. In Fig. 8.17, strengths are shown relative to the strength of a 1 : 3 cement-sand mortar and the brickwork built with it.

Optimum mortar mixes from considerations of maximum strength of brickwork for various strength of bricks, are suggested in Table 8.10, for general guidance of designers.

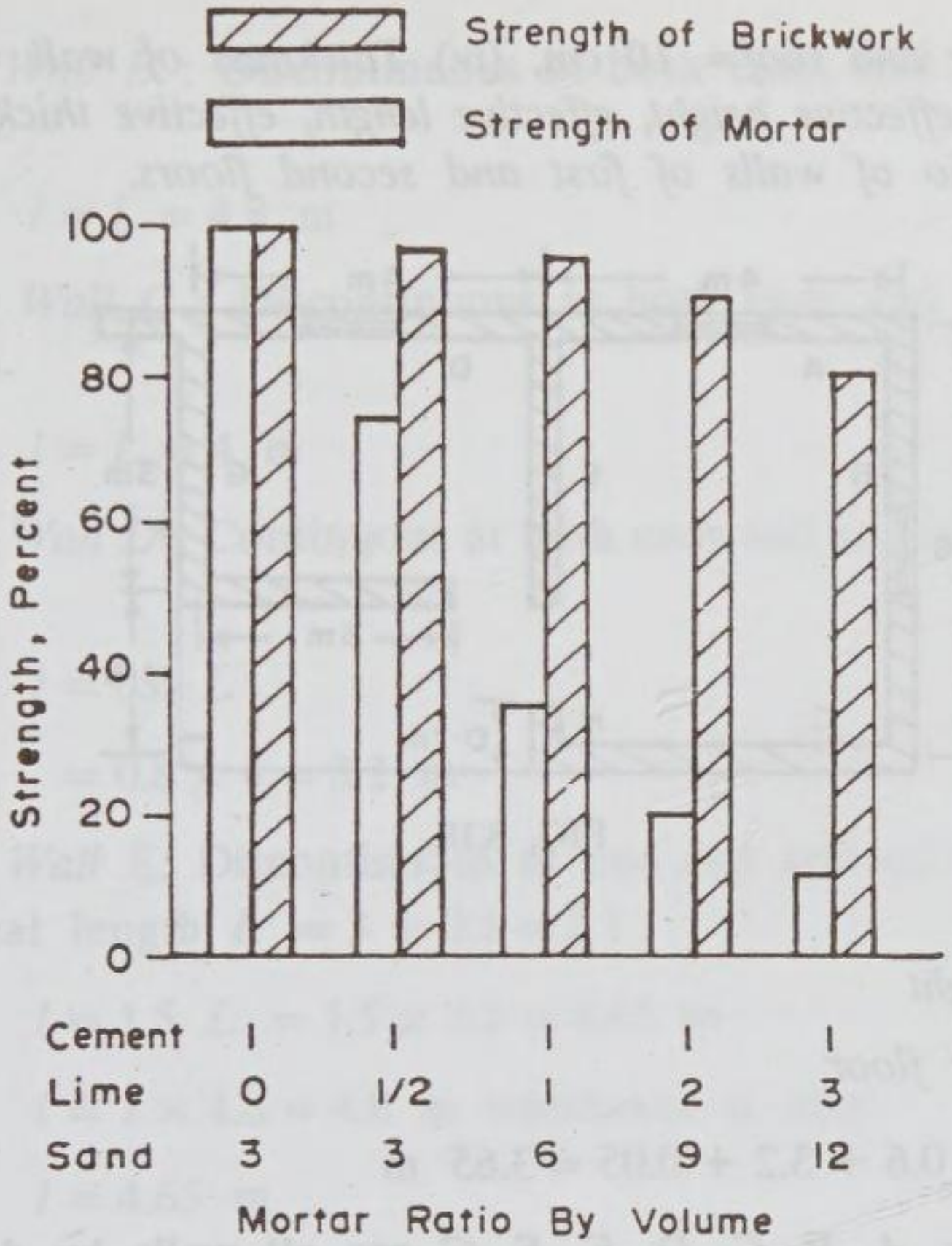


FIG. 8.17. EFFECTS OF MORTAR MIX PROPORTIONS (MORTAR RATIO BY VOLUME)

TABLE 8.10
OPTIMUM MORTAR MIXES FOR MAXIMUM STRENGTH WITH BRICKS OF VARIOUS STRENGTHS

	Brick Strength (kg/cm^2)	*Mortar Mix (By Volume)* *Cement : Lime : Sand*	*Mortar Type*
1.	Below 50	1 : 0 : 6 1 : 2 C : 9 0 : 1 A : 2–3	M2
2.	50 – 149	1 : 0 : 5 1 : 1 C : 6	M1
3.	150 – 249	1 : 0 : 4 1 : $\frac{1}{2}$ C : $4\frac{1}{2}$	H2
4.	250 or above	1 : 0 – $\frac{1}{4}$C : 3	H1

Note : $10\ kg/cm^2 = 1\ N/mm^2$

Example 8.1. *Fig. 8.18 shows the part plan of double storey house with the following data : (i) Clear height of floors = 3.2 m, (ii) Height of plinth above foundation footing = 0.6 , (iii) thickness*

of R.C.C. floor and roof= 10 cm, (iv) Thickness of wall = 20 cm. Determine the effective height, effective length, effective thickness and slenderness ratio of walls of first and second floors.

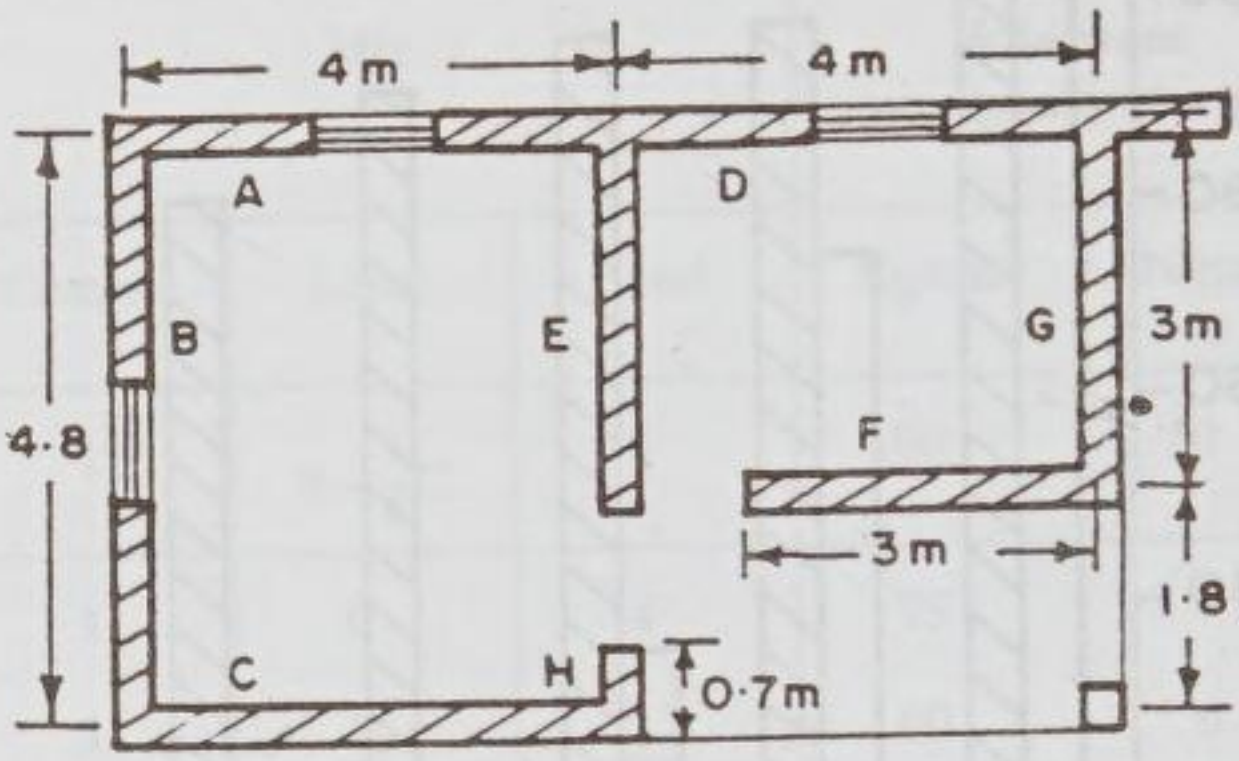


FIG. 8.18.

Solution.

1. Effective Height

(*a*) *First floor*

$$H = 0.6 + 3.2 + 0.05 = 3.65 \text{ m}$$

Elements *A, B, C, D, E, F, G* are all walls by definition, since their length is more than 4 times thickness; hence

$$h = 0.75 \text{ H} \qquad \text{(Fig. 8.4 } a\text{)}$$
$$= 0.75 \times 3.65 = 2.74 \text{ m.}$$

Element *H* has length less than four times the thickness : however, it is not a column since it is supported on one side by cross-wall. Hence its effective height is also found as in the case of wall, and will be equal 2.74 m.

(*b*) *Second floor*

$$H = 0.05 + 3.2 + 0.05 = 3.3 \text{ m}$$

Hence for all elements, $h = 0.75 \ H = 0.75 \times 3.3 = 2.48$ m.

2. Effective Length

Effective length will be the same for each floor.

(*i*) *Wall A* : Continuous at one end and discontinuous at the other, and is supported by cross-walls.

$\therefore$ $l = 0.9 L$ (Fig. 8.6)

$= 0.9 \times 4 = 3.6$ m.

(*ii*) *Wall B* : Discontinuous at both ends, and supported by cross-walls.

$\therefore$ $l = L = 4.8$ m (Fig. 8.7)

(*iii*) *Wall C* : Discontinuous at both ends and supported by cross-walls.

$\therefore$ $l = L = 4$ m (Fig. 8.7)

(*iv*) *Wall D* : Continuous at both ends and supported by cross-walls.

$\therefore$ $l = 0.8\ L$ (Fig. 8.5)

$= 0.8 \times 4 = 3.2$ m

(*v*) *Wall E*: Discontinuous at one end and opening at other end. Actual length $L = 3 + 0.1 = 3.1$

$l = 1.5\ L = 1.5 \times 3.1 = 4.65$ m (Fig. 8.10)

or $l = 1 \times 4.8 = 4.8$ m whichever is less. (Fig. 8.6)

$l = 4.65$ m

(*vi*) Wall *F* : Discontinuous at one end and opening at the other end.

$\therefore$ $l = 1.5\,L = 1.5 \times 3 = 4.5$ m (Fig. 8.10)

or $l = 1 \times L = 1 \times 4 = 4$ m whichever is less. (Fig. 8.7)

$\therefore$ $l = 4$

(*vii*) *Wall G* : Discontinuous at both ends

$\therefore$ $l = 3\,L$ (Fig. 8.9)

$= 2 \times 0.8 = 1.6$ m

3. *Effective Thickness*

Let us assume that joints are not raked. Actual thickness of wall = 19 cm, while nominal thickness is 20 cm. Those walls (such as *E, F, H*) which are not stiffened by cross-walls, effective thickness will be equal to actual thickness = 19 cm (*i.e.* $K_n = 1$). Walls, *A*, *B, C, D* and *G* are stiffened by cross-walls. Their stiffening co-efficients will be as under (Refer Table 8.3). The effective thickness of these walls will be equal to the actual thickness multiplied by their stiffening co-efficient.

For walls A, C and D, $\frac{t_P}{t_W} > 3$

$$\frac{S_P}{W_P} = \frac{4}{0.19} = 21$$

Since this is more than 20, $K_n = 1$ from Table 8.3.

Wall B : $\frac{t_P}{t\,w} > 3$; $\frac{S_P}{w_P} = \frac{4.8}{0.19} = 25$

$\therefore \qquad K_n = 1$

Wall *G* : $\frac{t_P}{t_W} > 3$; $\frac{S_P}{W_P} = \frac{3}{0.19} \approx 15$

$\therefore \qquad K_n = 1.2$

4. Slenderness Ratio (S_R)

The slenderness ratio of each wall is $\frac{h}{t \times K_n}$ or $\frac{l}{t}$ whichever is less. The values are tabulated in Table 8.11. The values of S_R shown in table is the one that is to be taken in consideration for design.

TABLE 8.11
SLENDERNESS RATIO

Wall	*First Floor (t = 0.19)*				*Second Floor (t = 0.19)*			
	h	*l*	K_n	S_R	*h*	*l*	K_n	S_R
A	2.74	3.6	1	14.4	2.48	3.6	1	13
B	2.74	4.8	1	14.4	2.48	4.8	1	13
C	2.74	4	1	14.4	2.48	4	1	13
D	2.74	3.2	1	14.4	2.48	3.2	1	13
E	2.74	4.32	1	14.4	2.48	4.32	1	13
F	2.74	3.6	1	14.4	2.48	3.6	1	13
G	2.74	3	1.2	12	2.48	3	1.2	10.9
H	2.74	1.6	1	14.4	2.48	1.6	1	13

Example 8.2. *A 20 cm thick brick wall carries an axial load of 5000 kg/m from wall above it and an eccentric load of 3600 kg/m from R.C.C. floor slab acting at a distance of 4.75 cm from the centre of the wall. Determine the equivalent eccentricity and stresses in the wall.*

Solution. (Fig. 8.19)

$W_1 = 5000$ kg/m

$W_2 = 3600$ kg/m

$e = 4.75$ cm

$t = 19$ cm

$$\bar{e} = \frac{W_2 . e}{W + W_2} \qquad ...(8.2)$$

$$= \frac{3600 \times 4.75}{5000 + 3600}$$

$$= \mathbf{1.99\ cm.}$$

∴ Equivalent eccentricity ratio

$$= \frac{\bar{e}}{t} = \frac{1.99}{19}$$

$$= 0.105$$

From Eqn. 8.3 (*a*)

$$f = \frac{W}{bt}\left(1 \pm \frac{6\,\bar{e}}{t}\right)$$

where $b = 1$ m$= 100$ cm ;

$W = 5000 + 3600$

$= 8600$

$$f = \frac{8600}{100 \times 19}(1 \pm 6 \times 0.15)$$

$= 4.53 \pm 2.84$

$= \mathbf{7.37\ kg/cm^2}$ **and** $\mathbf{1.69\ kg/cm^2}$

(*i.e.* both compressive)

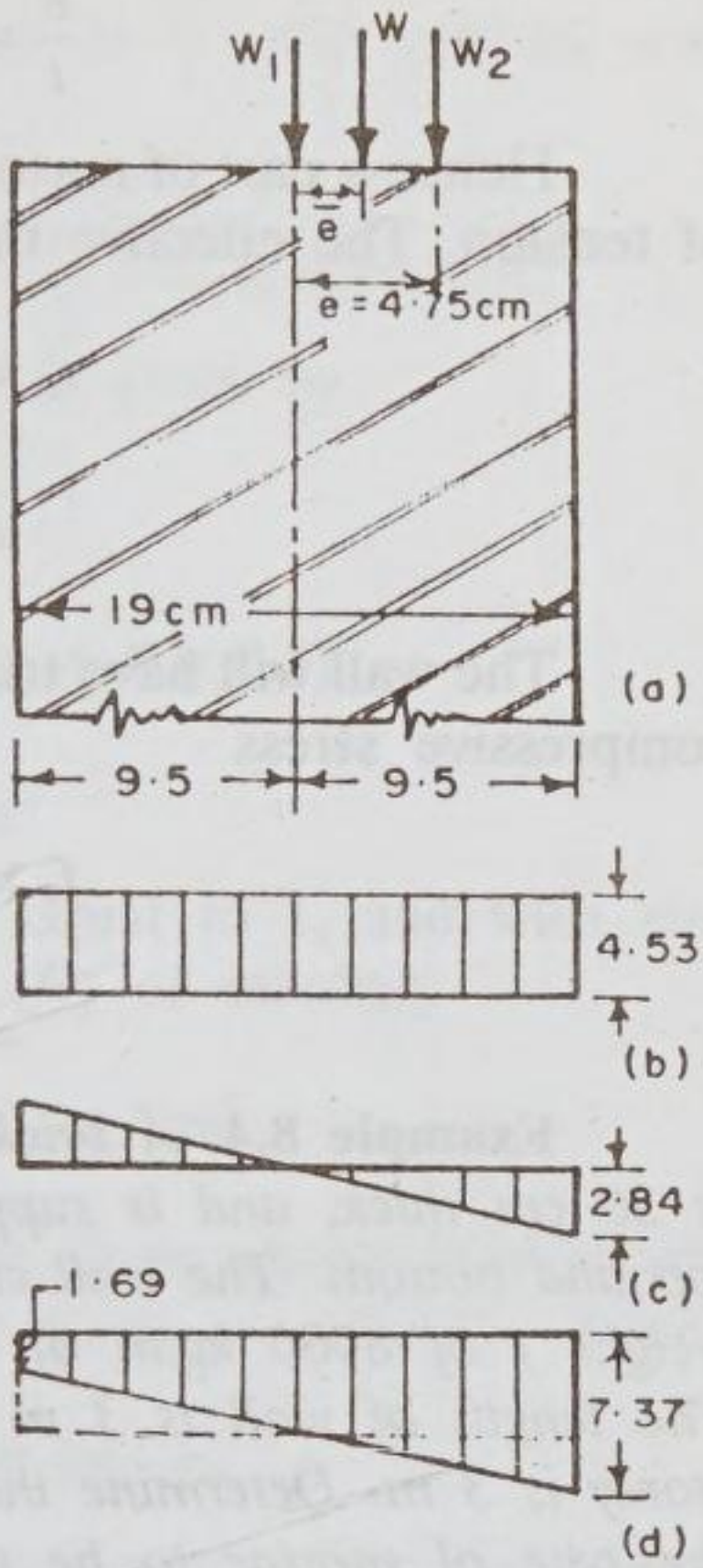


FIG. 8.19.

Example 8.3. *What will be the maximum compressive stress in wall of example 8.2 if* $W_1 = 3000$ *kg/m,* $W_2 = 6000$ *kg/m and* $e = 5$ *cm ?*

Solution. $\bar{e} = \dfrac{6000 \times 5}{3000 + 6000} = 3.33$ cm.

$$\frac{\bar{e}}{t} = \frac{3.33}{19} = 0.175 > \frac{b}{6}$$

Hence a part of masonry will be ineffective due to development of tension. The effective thickness of wall, resisting tension will be

$$t_e = 3\left(\frac{t}{2} - \bar{e}\right)$$

$$= 3\left(\frac{19}{2} - 3.33\right) = 18.51 \text{ cm.}$$

The wall will have triangular stress distribution, with maximum compressive stress

$$f_c = \frac{2W}{b \times t_e} = \frac{2 \times 9000}{100 \times 18.51}$$

$$= \mathbf{9.72\ kg/cm^2}$$

Example 8.4. *A brick masonry wall of a single room building is 20 cm thick, and is supported by 10 cm thick R.C.C. slab at its top and bottom. The wall carries a vertical load (inclusive of its own weight) of 8000 kg/m at the base, at an eccentricity ratio of 0.1. The length of wall is 3 m between cross-walls. The clear height of storey is 3 m. Determine the required crushing strength of bricks and the type of mortar to be used. Use modular bricks.*

Solution. $H = 3 + 0.1 = 3.1$ m ;

$L = 3$ m

$h = 0.75\ H = 0.75 \times 3.1 = 2.325$ m (Fig. 8.4 *a*)

$t = 0.19$ m

$l = L = 3$ m (Fig. 8.7)

$$\frac{t_P}{t_W} > 3\ ;\ \frac{S_P}{W_P} = \frac{3}{0.19} \approx 15$$

$\therefore$ Stiffening co-efficient $K_n = 1.2$ (Table 8.3)

Hence $$S_R = \frac{h}{t \times K_n} = \frac{2.325}{0.19 \times 1.2} = 10.2$$

or $$S_R = \frac{l}{t} = \frac{3}{0.19} = 15.8,$$ whichever is less.

Hence $S_R = 10.2$

$$\frac{\bar{e}}{t} = 0.1$$

Hence from Table 8.6, stress factor for $\frac{\bar{e}}{t} = 0.1$ and $S_R = 10.2$ is

$$K_S = 0.83 - \frac{0.83 - 0.74}{2} \times 0.2 \approx 0.82$$

Compressive stress in masonry, is given by

$$f_c = \frac{W}{bt}\left(1 + \frac{6\bar{e}}{t}\right)$$

$$= \frac{8000}{100 \times 19}(1 + 6 \times 0.1)$$

$$= 6.74 \text{ kg/cm}^2$$

With shape modification factor equal to 1, and with stress factor K_S=0.82, required basic stress (f_b) of masonry

$$= \frac{6.74}{0.82} = 8.22 \text{ kg/cm}^2$$

Because of eccentric loading, the Code allows 25% increase in the permissible stress. Hence basic stress of requisite masonry

$$= \frac{8.22}{1.25} = 6.58 \text{ kg/cm}^2$$

From Table 8.5, we find that brick of 70 kg/cm^2 strength will be required. From Table 8.8, the shape of modification factor for modular bricks (having height to width ratio equal to 1) will be 1.1.

Thus, basic stress required $= \frac{6.58}{1.1} \approx 6 \text{ kg/cm}^2$.

Referring to Table 8.5 again, the following masonry will be required:

Bricks: 70 kg/cm^2 strength

Mortar : M 1 (*i.e.* 1 : 1 : 6)

Hence the masonry required will be 70-M 1

Example 8.5. *Redesign the wall of example 8.4 if the load is perfectly axial, and if the length of the wall is 1.2 m only without any cross- walls.*

Solution. Stiffening co-efficient will be equal to unity.

$$\therefore \quad S_R = \frac{h}{t} = \frac{2.325}{0.19} = 12.2$$

Hence from Table 8.6, stress factor for $\frac{\bar{e}}{t} = 0$ and $S_R = 12.2$ is

$$K_S = 0.76 - \frac{0.76 - 0.67}{2} \times 0.2 \approx 0.75$$

Area of wall in plan $= 19 \times 120 = 2280\ \text{cm}^2$

$\therefore$ Area of reduction factor

$$K_a = 0.75 + \frac{A}{12000}$$

$$= 0.75 + \frac{2280}{12000}$$

$$= 0.75 + 0.19 = 0.94.$$

Hence basic stress of requisity masonry, with unit shape factor is

$$= \left(\frac{8000}{100 \times 19}\right) \times \frac{1}{0.75 \times 0.94} = 5.97\ \text{kg/cm}^2.$$

From Table 8.5, we find that bricks of $70\ \text{kg/cm}^2$ strength will be required. From Table 8.8, the shape modification factor for modular bricks will be 1.1

$$\therefore \qquad f_b \text{ required} = \frac{5.97}{1.1} = 5.43\ \text{kg/cm}^2$$

Referring to Table 8.5 again, the following masonry will be required :

Bricks : $70\ \text{kg/cm}^2$ strength

Mortar : M 2 (*i.e.* 1 : 2 : 9 or 1 : 6)

Hence masonry required will be **70–M 2.**

Example 8.6. *Fig. 8.20 shows the plan of a room of a single-storeyed house having clear height of 3 m. The height of plinth is 1.5 m above foundation footing. The R.C.C. roof slab has thickness of 10 cm, with clear span of 3 m, bearing on the front wall AB. The height of parapet above roof is 0.9 which is plastered on both sides. The brick wall is plastered from inside and has raked joints. Design the wall AB. The roof carries a live load of 150 kg/m^2. Take unit weight of masonry as 2000 kg/m^2. Height of door opening == 2 m.*

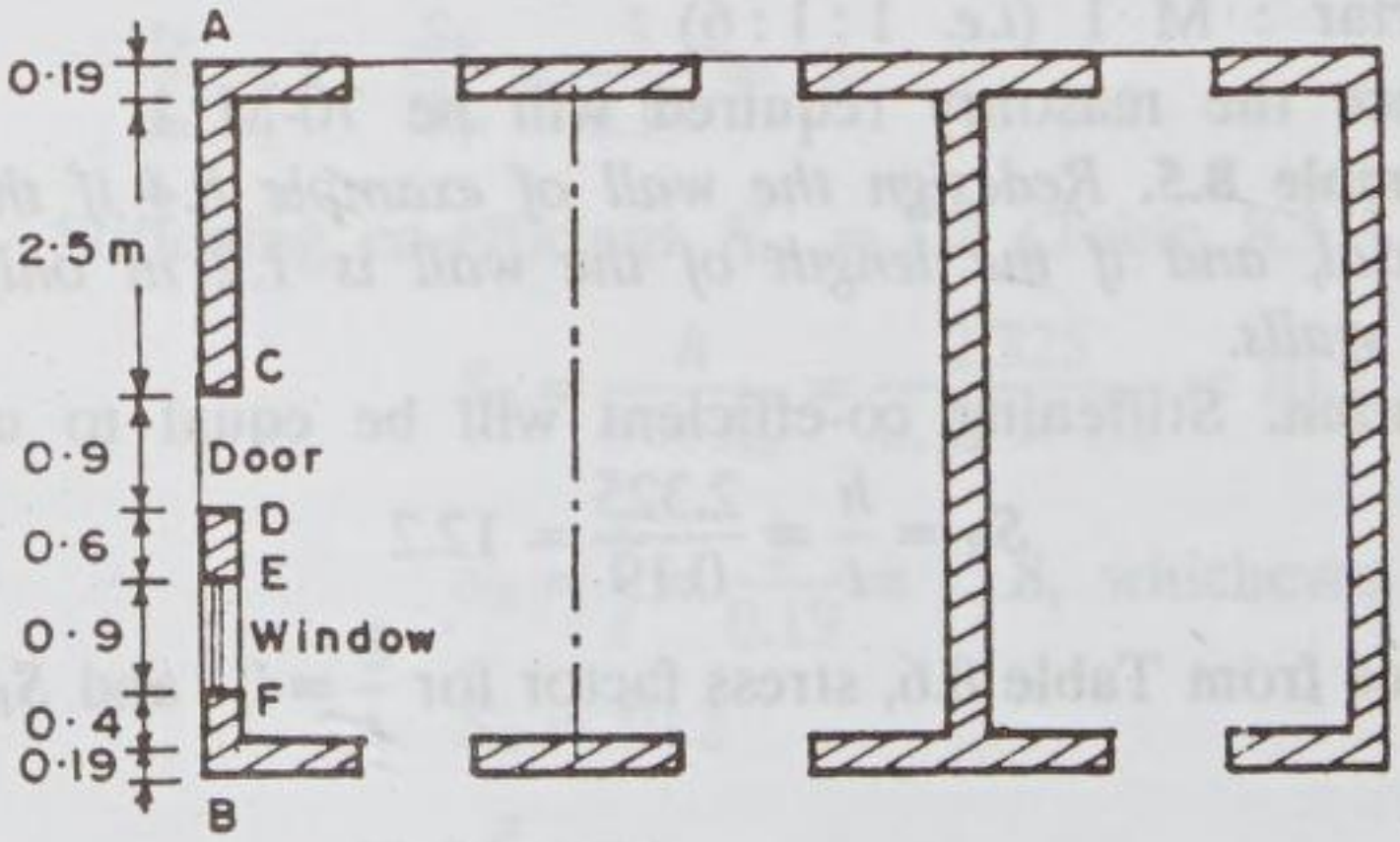


FIG. 8.20.

Solution. By inspection, elements *DE* and *FB* of the wall will have the maximum stress. Let us work out the stresses at plinth level.

$$\textit{Loads} \ : \ \text{Parapet} = \left(\frac{19+3}{100}\right) \times 0.9 \times 2000 \quad = 396 \ \text{kg/m.}$$

$$\text{Wall} = \frac{19+1.5}{100} \times 3 \times 2000 \qquad = 1230 \ \text{kg/m}$$

$$\text{Total} \qquad = 1626 \ \text{kg/m}$$

(It is common practice not to make any deductions for opening since calculations for the design of masonry are not very precise.).

Roof Load : R.C.C. slab $= 10 \times 25 = 250 \ \text{kg/m}^2$

Lime concrete terrace 10 cm thick $= 10 \times 20 = 200 \ \text{kg/m}^2$

Live load $= 150 \ \text{kg/m}^2$

$\therefore$ Total Roof load $= 600 \ \text{kg/m}^2$

Effective span of slab $= 3.0 + 0.1 = 3.1 \ \text{m}$

$$\therefore \ \text{Roof load on wall} = \frac{600 \times 3.1}{2} = 930 \ \text{kg/m}$$

Portion FB of Wall

$$\text{Length of wall} \quad = 0.4 \ \text{m} + \frac{0.19}{2} \approx 0.5 \ \text{m.}$$

Though this comes under the definition of column, we will treat it as wall because of stiffening by cross-wall. Due to this, no area reduction factor is applicable.

The wall will carry additional load due to window opening.

$$\therefore \ \text{Total load} = (1626 + 930)\left(0.5 + \frac{0.9}{2}\right) = 2428 \ \text{kg/m}$$

Because of raked joints, $t = 19 - 1 = 18$ cm

$$\therefore \ \text{Compressive stress} = \frac{2428}{18 \times 50} = 2.7 \ \text{kg/cm}^2$$

$$\text{Slenderness ratio} \quad = \frac{h}{t} = \frac{(1.5 + 3 + 0.05)}{0.18} \times 0.75 \approx 19$$

$$\text{Effective length} \quad l = 2L = 2 \times 0.5 = 1 \ \text{m}$$

$$S_R = \frac{1.0}{0.18} = 5.6 \approx 6$$

$$\text{Hence governing} \quad = S_R = 6$$

From Table 8.6, stress factor (for $S_R = 6$) is 1.00

Hence basic stress required $= \frac{2.7}{1} = 2.7 \text{ kg/cm}^2$

From Table 8.5, we find that bricks of 35 kg/cm^2 could be used. Shape modification factor for this strength is 1.2 (Table 8.8).

∴ Requisite basic stress of masonry.

$$= \frac{2.7}{1.2} = 2.25 \text{ kg/cm}^2$$

Portion DE of wall

Length $= 0.6$ m.

This wall carries load of both the openings to its either side.

∴ Load on wall $= (1626 + 930)\left[\frac{0.9}{2} + 0.6 + \frac{0.9}{2}\right]$

$= 3834$ kg.

∴ Compressive stress (f) at plinth level

$$= \frac{3834}{18 \times 60} = 4.26 \text{ kg/cm}^2$$

This portion of wall comes under the definition of a column.

∵ Effective height $= 1.5 \times$ height of taller opening

$= 1.5 \times 2 = 3$ m.

Effective height of wall without opening

$= 0.75\ H$

$= 0.75\ (1.5 + 3 + 0.05) = 3.41$ m.

Effective height of wall without opening

$= 0.75\ H$

$= 0.75\ (1.5 + 3 + 0.05) = 3.41$ m.

Effective height of wall taken as column

$= H = 1.5 + 3 + 0.05 = 4.55$ m.

Effective height $= 3$ m.

$$S_R = \frac{3}{0.18} = 16.7$$

Stress factor K_S (Table 8.6) for $S_R = 16.7$ is

$$K_S = 0.58 - \frac{0.58 - 0.5}{2} \times 0.7 = 0.55$$

Area of wall in plan $= 18 \times 60 = 1080 \text{ cm}^2$

Area reduction factor $K_a = 0.75 + \frac{1080}{1200}$

$= 0.84.$

Hence basic stress of requisite masonry with unity shape modification factor

$$= \frac{f}{K_s \times K_a} = \frac{4.26}{0.55 \times 0.84} = 9.22 \text{ kg/cm}^2$$

Thus this portion of wall carries maximum stress, and will govern the design.

From Table 8.5, we find that bricks of 105 kg/cm^2 strength will be required. The shape reduction factor of modular bricks of this stress is 1.1 from Table 8.8.

$$\therefore f_b \text{ required} = \frac{9.2}{1.1} = 8.36 \text{ kg/cm}^2$$

From Table 8.5, we find that masonry should have bricks of 105 kg/cm^2 strength, with M 2 mortar (1 : 2 : 9 or 1 : 6), giving a basic strength of 8.5 kg/cm^2.

Hence required masonry is **105–M2.**

PROBLEMS

1. Explain how do you determine (*a*) Effective length, and (*b*) Effective height of a masonry wall.

2. What do you understand by slenderness ratio ? How do you determine it ?

3. What do you understand by lateral support to a wall ? what is its function ?

4. Write notes on (*i*) stiffening co-efficient, (*ii*) area reduction factor, (*iii*) stress factor, (*iv*) basic compressive stress, and (*v*) shape modification factor.

5. Explain how do you use nomograms for the design of a masonry wall.

6. Explain, step by step, the analytical method of designing a masonry wall.

7. Write a note on 'selection of mortar' for masonry walls.

8. A 30 cm thick brick wall carries an axial load of 8000 kg/m and an eccentric load of 5000 kg/m at an eccentricity of 7.5 cm from the centre of wall thickness. Determine maximum compressive stress in the masonry.

9. A 30 cm thick masonry wall of a multi–storey building is supported by 15 cm thick R.C.C. slab at its top and bottom, and carries an axial load of 15000 kg/m at the base. The length of wall is 3.2 m between cross-wall.

The wall is continuous beyond the two cross-walls. The clear height of storey is 3 m. Determine the required crushing strength of bricks and the type of mortar to be used.

10. Redesign the wall of problem 9, if it carries a vertical load of 10000 kg/m, inclusive of its own weight, at an eccentricity ratio of 0.07.

11. A short wall of 1m length and 20 cm thickness carries an axial load of 10000 kg/m. The wall is free at both ends. The height of the wall from the level of foundation footing to the centre of roof slab is 4.2 m. Design the masonry for the wall.

12. Design the masonry of wall *AB* of the room shown in Fig. 8.21. The clear height of wall is 3.2 m, while the height of plinth above foundation footing is 1.6 m. The thickness of wall is 20 cm. The roof carries a live load of 150 kg/m^2, and has a parapet of 1 m high.

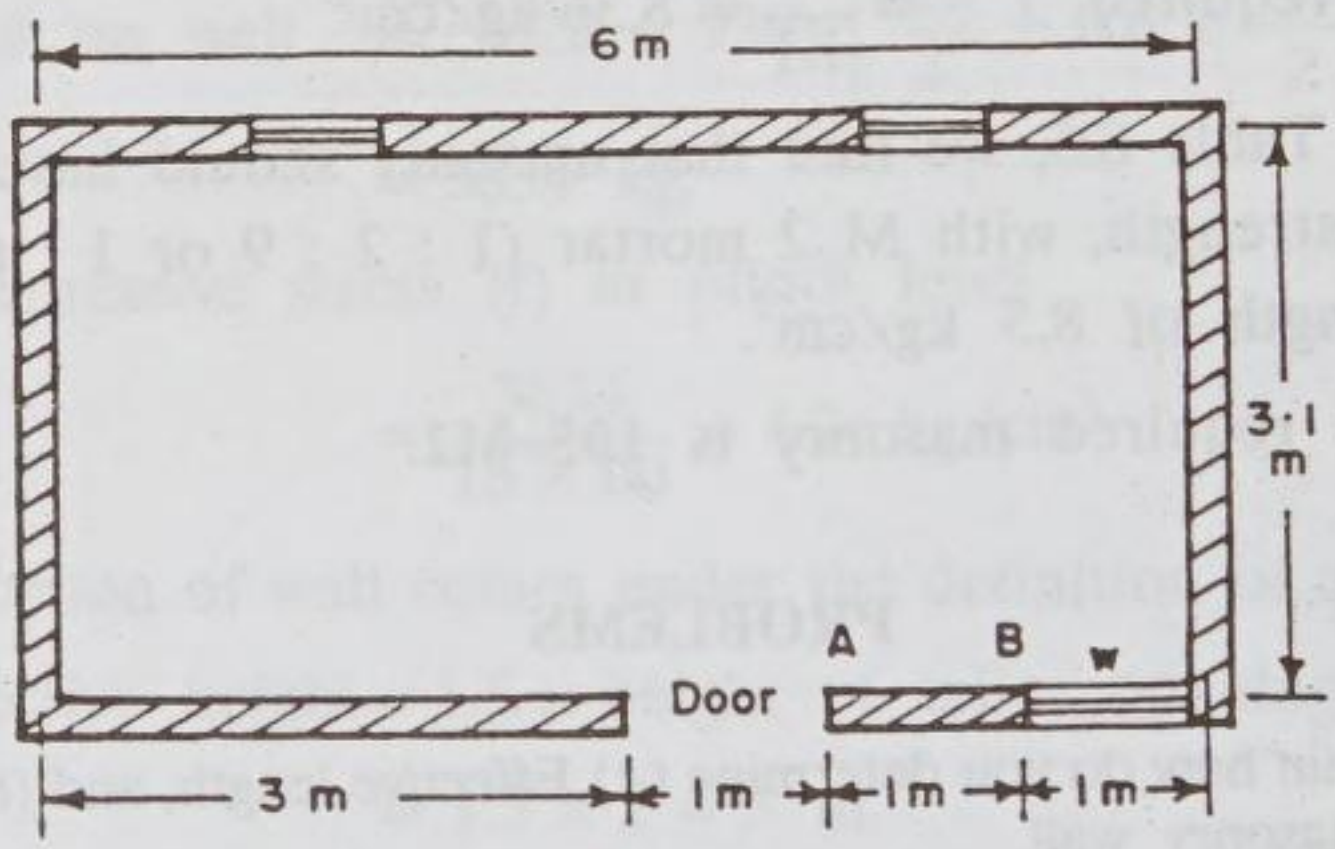


FIG. 8.21

9

Cavity Walls

9.1. INTRODUCTION

A *cavity wall or hollow wall* is the one which consists of two separate walls, called *leaves or skins*, with a *cavity or gap* in-between. The two leaves of a cavity wall may be of equal thickness if it is a non-load-bearing wall, or the internal leaf may be thicker than the external leaf, to meet the structural requirements. The two portions of the wall may be connected together by metal pins or bonding bricks at suitable interval. Cavity walls are often constructed for giving better thermal insulation to the building. It also prevents the dampness to enter and acts as sound insulation. Thus they are *normally* the outer walls of the building. The size of cavity varies from 4 to 10 cm. The inner and outer skins should not be less than 10 cm each (half brick).

Advantages

Cavity walls have following advantages over other walls.

1. There is no direct contact between the inner and outer leaves of the wall(except at the wall ties). Hence the external moisture(dampness) cannot travel inside the building.
2. The cavity between the two leaves is full of air which is bad conductor of heat. Hence transmission of heat from external face to the inside the room is very much reduced. Cavity walls have about 25% greater insulating value than the solid walls.
3. Cavity walls also offer good insulation against sound.
4. The nuisance of efflorescence is also very much reduced.
5. They are cheaper and economical.
6. Loads on foundations are reduced because of lesser solid thickness.

9.2. GENERAL FEATURES OF CAVITY WALLS

Fig. 9.1 shows the vertical sections of various types of cavity walls for flat and inclined roofs. In the case of brick cavity wall, each leaf is half brick thick. Such a wall is capable of taking load

of two storeyed building of the domestic type. However, if heavier loads are to be supported, the thickness of inner leaf can be increased in the multiple of half brick thickness. The cavity should neither be less than 40 mm nor more than 100 mm in width. The inner and outer skins are adequately tied together by means of special wall ties placed in suitable arrangement, at the rate of atleast five ties to a square metre of wall area. According to Building Regulations of U.K., the ties must be placed at distances apart not exceeding

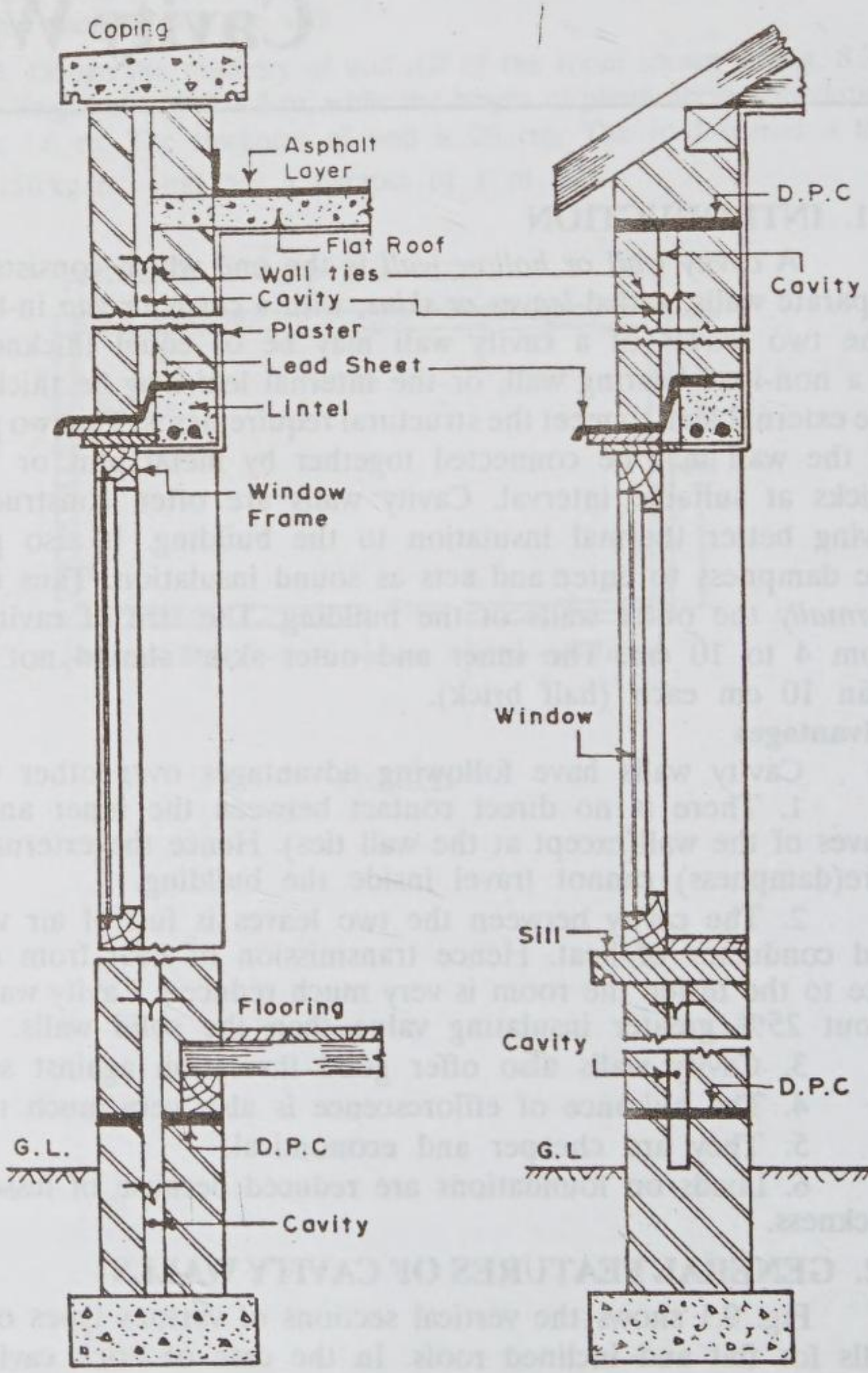


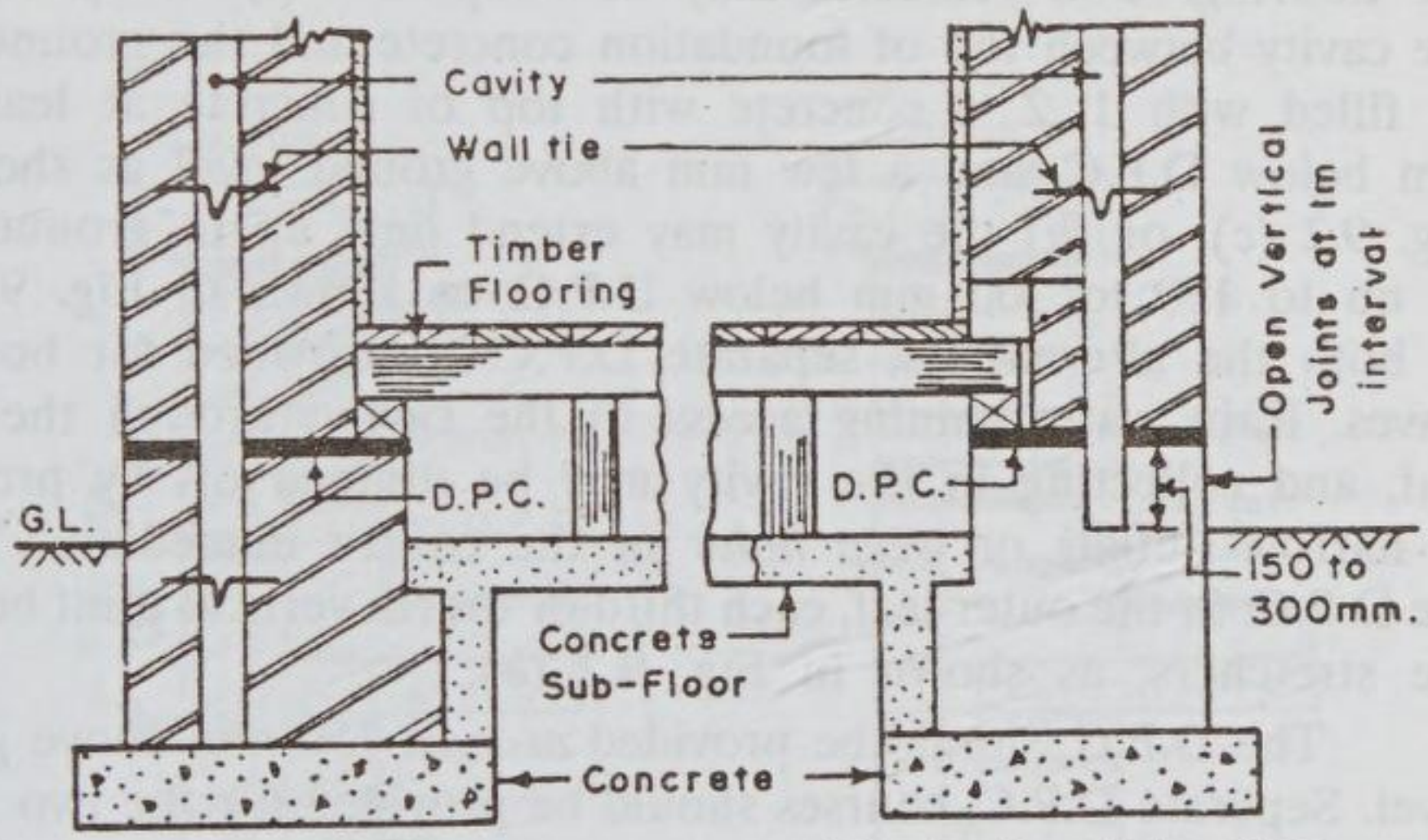
FIG. 9.1. BRICK CAVITY WALLS.

900 mm horizontally and 450 mm vertically. The ties are staggered. Ties must be placed at 300 mm vertical intervals at all angles and doors and window jambs to increase stability.

Since the cavity separates the two leaves of the wall, to prevent moisture to enter, it is essential to provide a vertical damp proof course at window and door reveals. The damp proof course should be flexible.

9.3. POSITION OF CAVITY AT FOUNDATION LEVEL

The cavity extends vertically all along the height of the wall, except at the openings, where it is discontinued. At the top of the wall, it extends *upto* coping in the case of flat roofs with parapet



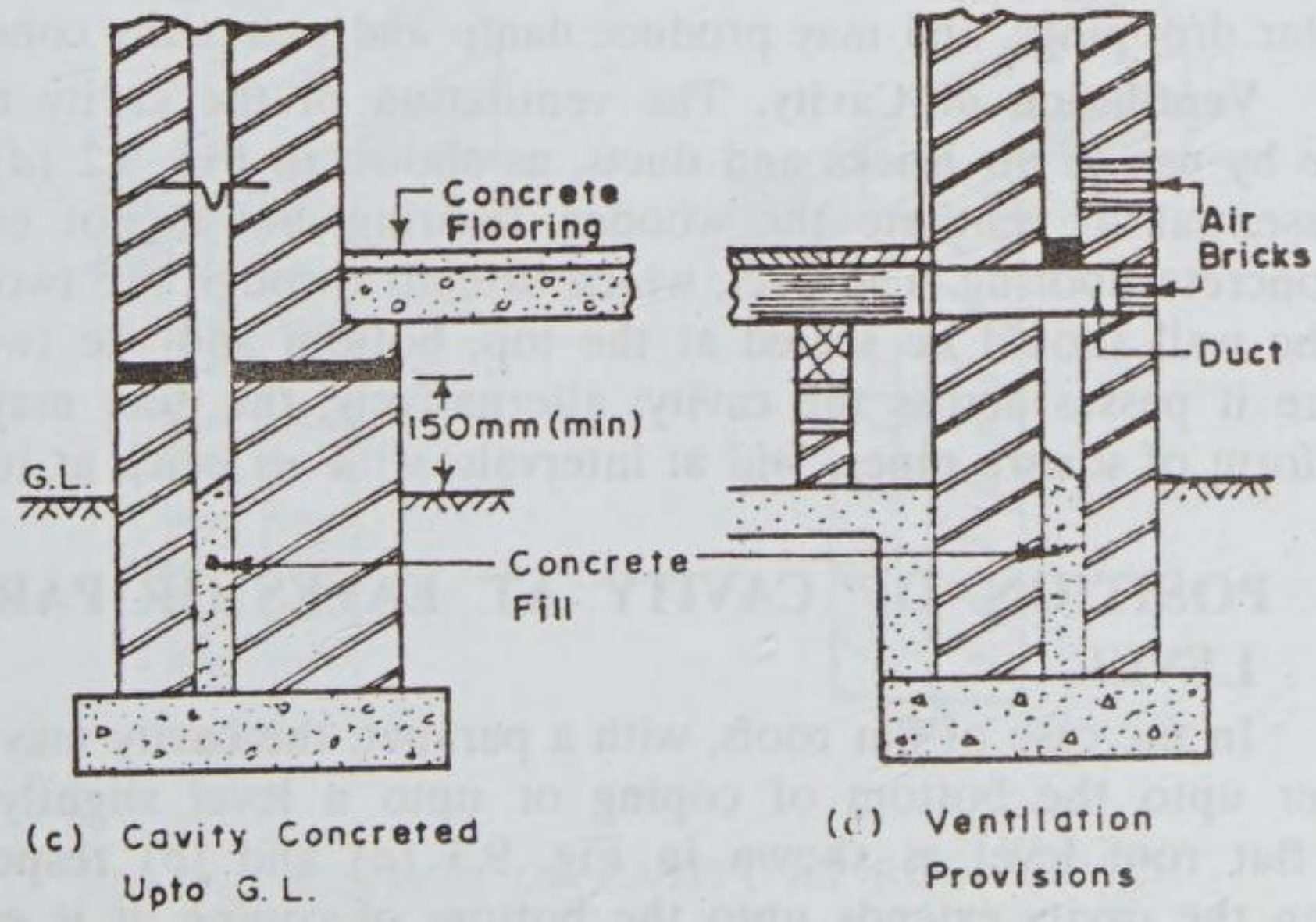


FIG. 9.2. POSITION OF CAVITY AT FOUNDATION LEVEL.

wall and upto or near eaves level in the case of sloping roof. In the foundations, the cavity may either extend *upto* concrete base or upto 15 to 30 cm below the damp proof course. Fig. 9.2 shows various alternative positions of bottom of cavity.

Fig. 9.2 (*a*) shows the cavity extending right upto the concrete base of the footing, with damp-proof-course (D.P.C.) introduced just below the floor level. This is a more common arrangement. However, if the brick work below ground level is not carefully constructed, specially in the areas where soil water level is high, water will enter through the joints and will collect in the cavity. This water will further travel through the inner leaf, and will cause dampness in the flooring. Two remedies may be adopted : (*i*) the portion of the cavity between top of foundation concrete and the ground level be filled with 1 : 2 : 4 concrete with top of concrete at least 150 mm below D.P.C. and a few mm above ground level as shown in Fig. 9.2 (*c*), or (*ii*) the cavity may extend only up to ground level or up to 150 to 300 mm below D.P.C. as shown in Fig. 9.2 (*b*). In both the alternatives, separate D.P.C. is provided for both the leaves. Rain water gaining access to the cavity through the outer leaf, and collecting in the cavity may be drained off by provision of narrow outlets or *weep holes* in the course immediately below the D.P.C. in the outer leaf, each third or fourth vertical joint between the stretchers, as shown in Fig. 9.2 (*b*).

The D.P.C. should be provided *at least* 150 mm above ground level. Separate D.P.C. courses should be provided for the two leaves. The cavity should extend below the D.P.C. level atleast by 150 mm. If the bottom of the cavity is level with D.P.C., or if the D.P.C. is provided over the full width of wall (*i.e.*, bridging over the air gap), water may be conducted to the inner leaf through accumulated mortar droppings, and may produce damp and unhealthy conditions.

Ventilation of Cavity. The ventilation of the cavity may be done by use of air bricks and ducts, as shown in Fig. 9.2 (*d*). Duct is essential to ventilate the wooden flooring, but is not essential in concrete flooring. The duct, which extends through the two leaves of the wall should be sealed at the top, bottom and the two sides where it passes across the cavity; alternatively, the duct may be in the form of square pipes, laid at intervals, with air brick at its outer end.

9.4. POSITION OF CAVITY AT EAVES OR PARAPET LEVEL

In the case of flat roofs, with a parapet, the cavity may extend either upto the bottom of coping or upto a level slightly above the flat roof level as shown in Fig. 9.3 (*a*) and (*b*) respectively. When the cavity extends upto the bottom of coping, it is essential

to have a D.P.C. course between the bottom of coping and top cavity, so that rain water does not enter the cavity. If the cavity is terminated just above the flat roof, one D.P.C. is provided over the top of the cavity and the other below the bottom of coping. In both the cases, it is *better* if a flexible D.P.C. and drip is provided, starting from roofing and bridging over the two leaves and cavity, as shown. This will check accumulation of rain water in the cavity, entering through the inner leaf forming the parapet. The water collected on the flexible D.P.C. may be drained out through providing open vertical joints(or weep holes) after every third or fourth vertical joint.

Figs. 9.3 (*c*) and (*d*) show the details of cavity at eaves level.

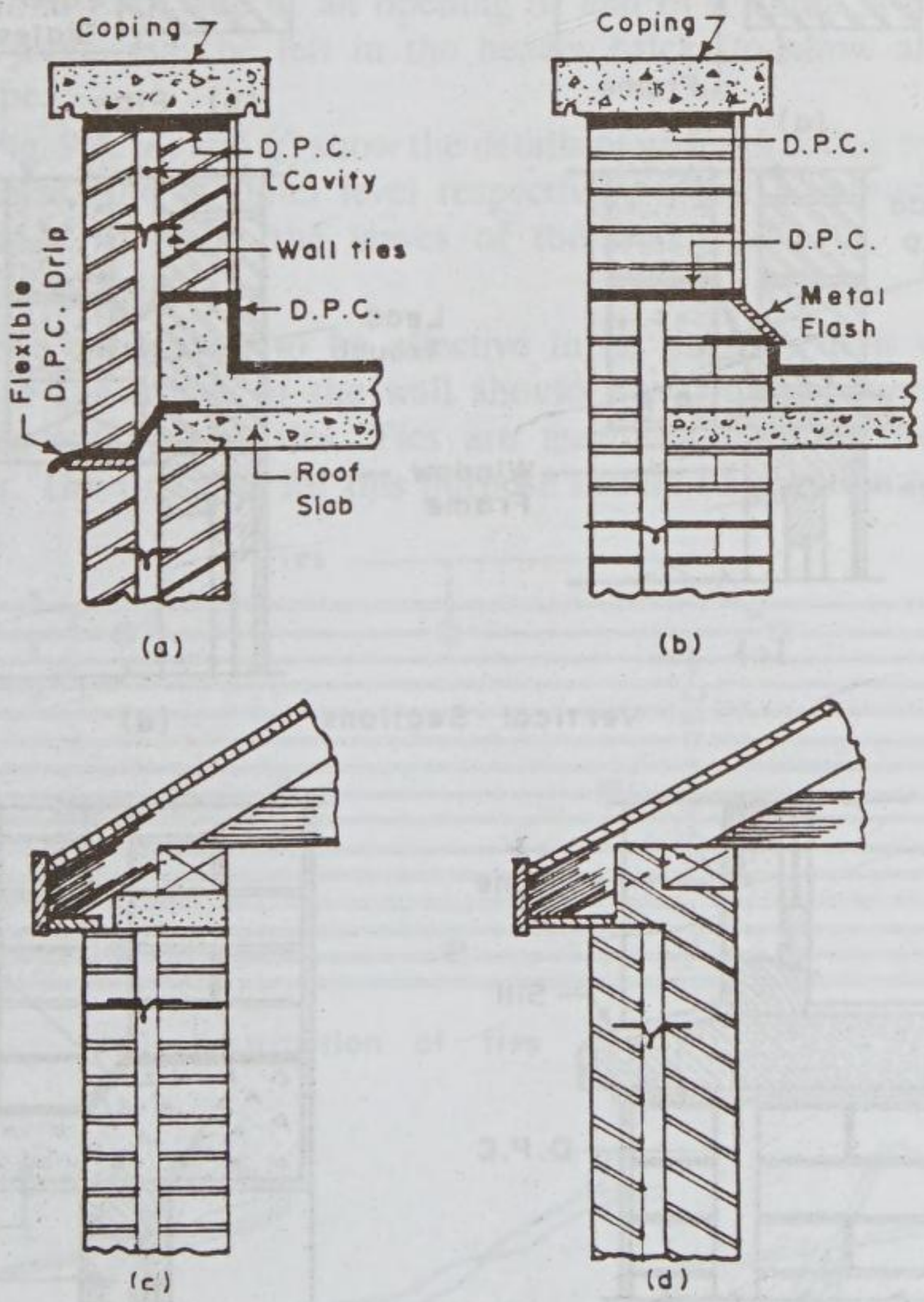


FIG. 9.3. POSITION OF CAVITY AT ROOF LEVEL.

9.5. CAVITY WALL AT OPENINGS

In the plan, the cavity is discontinued at the openings, such as doors, windows etc. Fig. 9.4 (*a*) and (*b*) show some details of location of damp resisting material. In Fig. 9.4 (*a*), the damp resisting material consists of a double layer of slates, bedded in cement, with the outer layer projecting in a groove in the frame. This groove is filled with oil mastic, as the work proceeds. In Fig. 9.4 (*b*), lead,

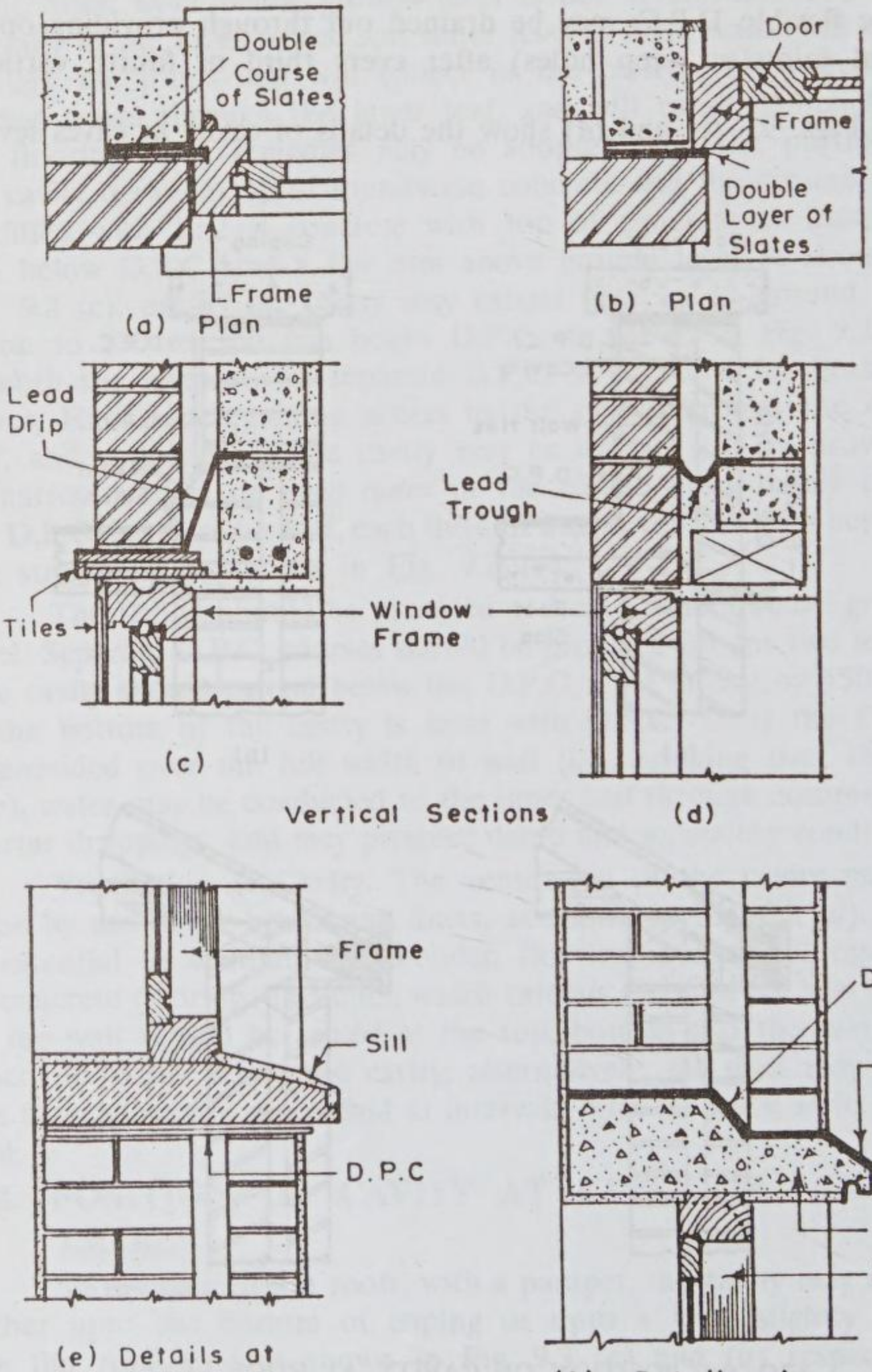


FIG. 9.4. DETAILS AT OPENINGS

asphalt felt or a double layer of slates is applied at the slightly recessed jamb. An alternative method consists of placing, as the work proceeds, a vertical layer of asphalt felt or lead in lieu of slates. This layer should be 215 mm wide, extending to the groove of the window frame at one edge and into the cavity at the other.

At the top of the opening separate lintels (or arches) should preferably be provided for each leaf, so that cavity is continuous. Proper protection against dampness is essential because the water passing through defective joints etc. in the outer leaf will travel down its inter face, come in contact with the lintel and will spread inside. The protection is provided in the form of lead, copper or asphalt felt covering, stepped down from the inner leaf, as shown in Fig. 9.4 (*c*) and (*d*). This covering should extend for 75 to 150 mm beyond each side of an opening or end of a lintel. A few open vertical joints may be left in the header bricks to allow any water to escape.

Fig. 9.4. (*e*) and (*f*) show the details of water proofing treatment at sill level, and at lintel level respectively, when a common lintel is provided for both the leaves of the wall.

9.6. WALL TIES

For cavity wall to be effective in its purpose, it is essential that both the leaves of the wall should not come in contact with other, except at wall ties. Ties are used to hold the two leaves together. The ties used for this purpose should be sufficiently strong,

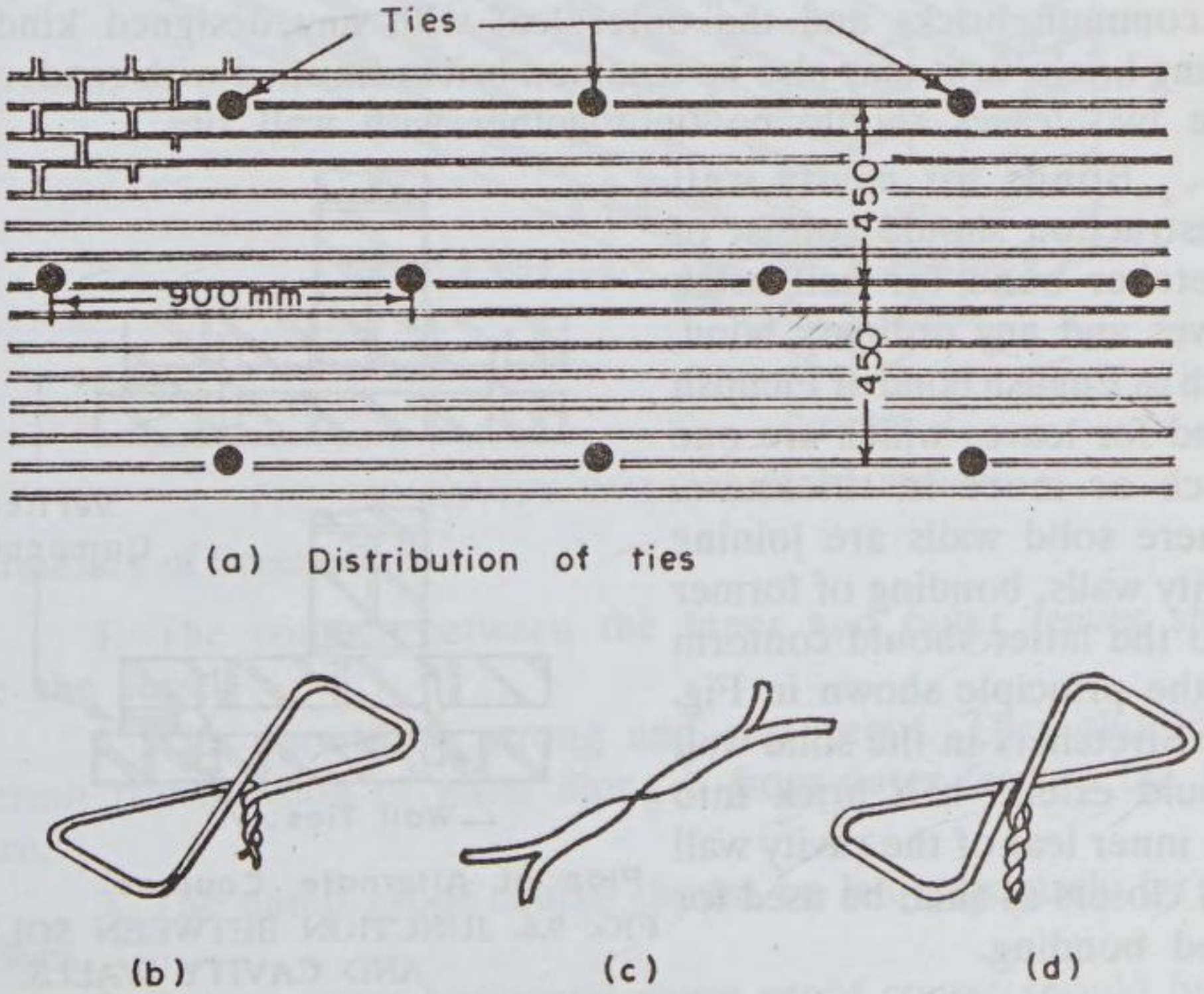


FIG. 9.5. WALL TIES.

be non-corrodible and should be so shaped that water entering through the outer leaf does not travel along it. These ties must be placed at distances not exceeding 900 mm horizontally and 450 mm vertically, and should be staggered as shown in Fig. 9.5.

Wall ties are usually made of mild steel, thoroughly galvanized or dipped in hot tar and sanded to protect them from rust. For important buildings or for buildings near the sea, copper or bronze or similar durable and highly corrosive- resistant metal is used for ties. Various forms of metal ties are shown in Figs. 9.5 (*b*), (*c*) and (*d*).

Fig. 9.5 (*b*) shows the wire tie, commonly used; the ends are twisted and turned down, so that the moisture travelling along it drops down in the cavity. Also, the mortar droppings do not readily lodge on it because of the thinness of the wire. Wires may be of 3 to 4 mm dia. Fig. 9.5 (*d*) shows a similar tie, made out of flat bar section twisted at the end. Tie shown in Fig. 9.5 (*c*) has forked ends made out of flat bar, twisted in the middle. This tie is quite stiff and durable.

9.7. CONSTRUCTION OF CAVITY WALL

Generally, the cavity wall is set centrally over the concrete base, without any footings. According to I.S. recommendations, the lower portion of the cavity may be filled with lean concrete upto a few centimeters above the existing ground level. The top of the filling should be sloped (Fig. 9.2 *c, d*), with weep holes at 1 m intervals along the outer leaf of the wall. The inner leaf may be of common bricks and the outer leaf with any designed kind of facing bricks or it may also be common bricks finished with rendering. The two leaves should be tie together with wall ties.

Bonds for cavity wall construction should consist of stretcher bond for half brick leaves and any ordinary bond, such as English bond or Flemish bond for leaves which are one brick or more in thickness. Where solid walls are joining cavity walls, bonding of former into the latter should conform to the principle shown in Fig. 9.6. Stretchers in the solid wall should extend half brick into the inner leaf of the cavity wall and closers as shall be used for good bonding.

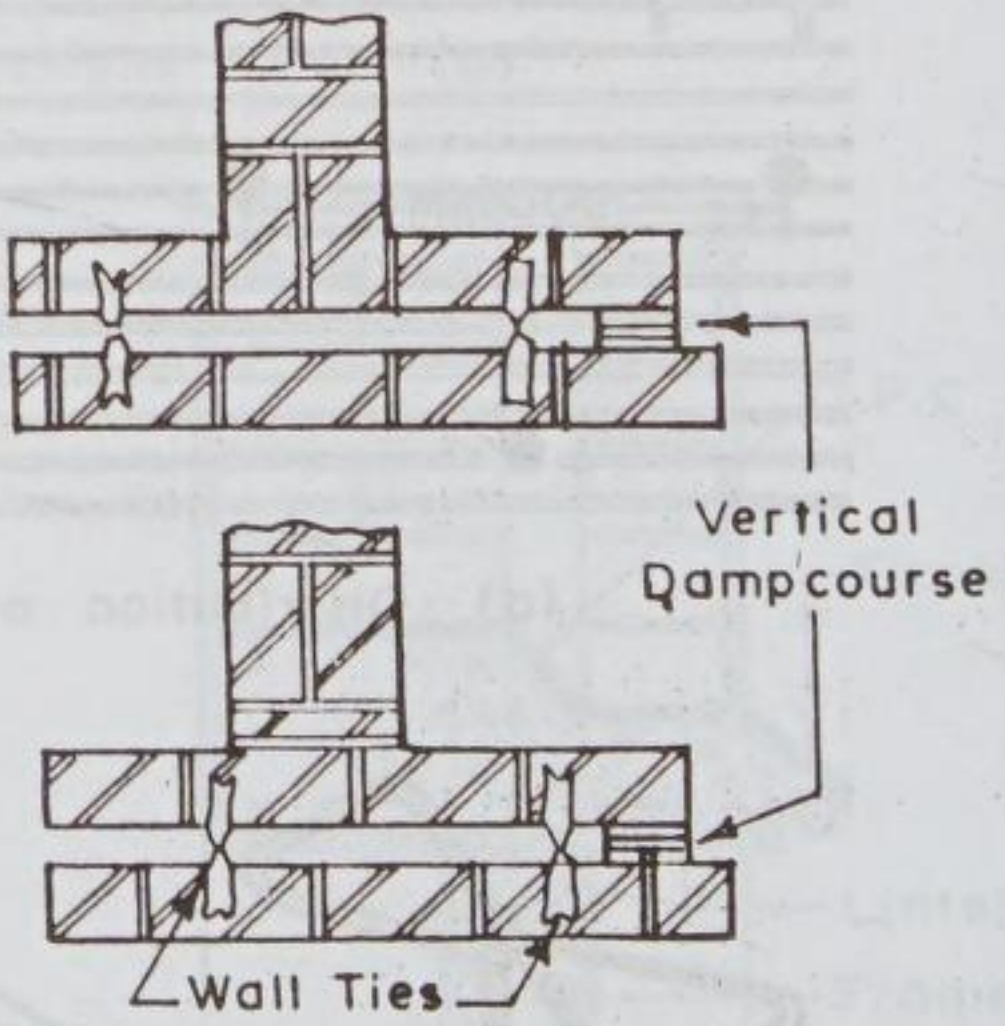


FIG. 9.6. JUNCTION BETWEEN SOLID AND CAVITY WALLS.

Bricks should be laid very carefully to leave the cavity free from mortar droppings. Two leaves of the wall should be raised simultaneously and uniformly. The position of wall ties should be predetermined so as to have uniform spacing preferably in centres. The cavity should be made free from rubbish and mortar droppings by means of a timber batten 25 mm thick and width about 12 mm less than the cavity, resting over the ties. The battens may be lifted by means of wires or rails attached to the battens, as shown in Fig. 9.7. The batten is supported on wall ties and the brick work is carried out on either side of the batten, to the height where next row of wall ties are to be provided. After this, the batten is lifted up, cleaned of mortar droppings and replaced over the next row of wall ties.

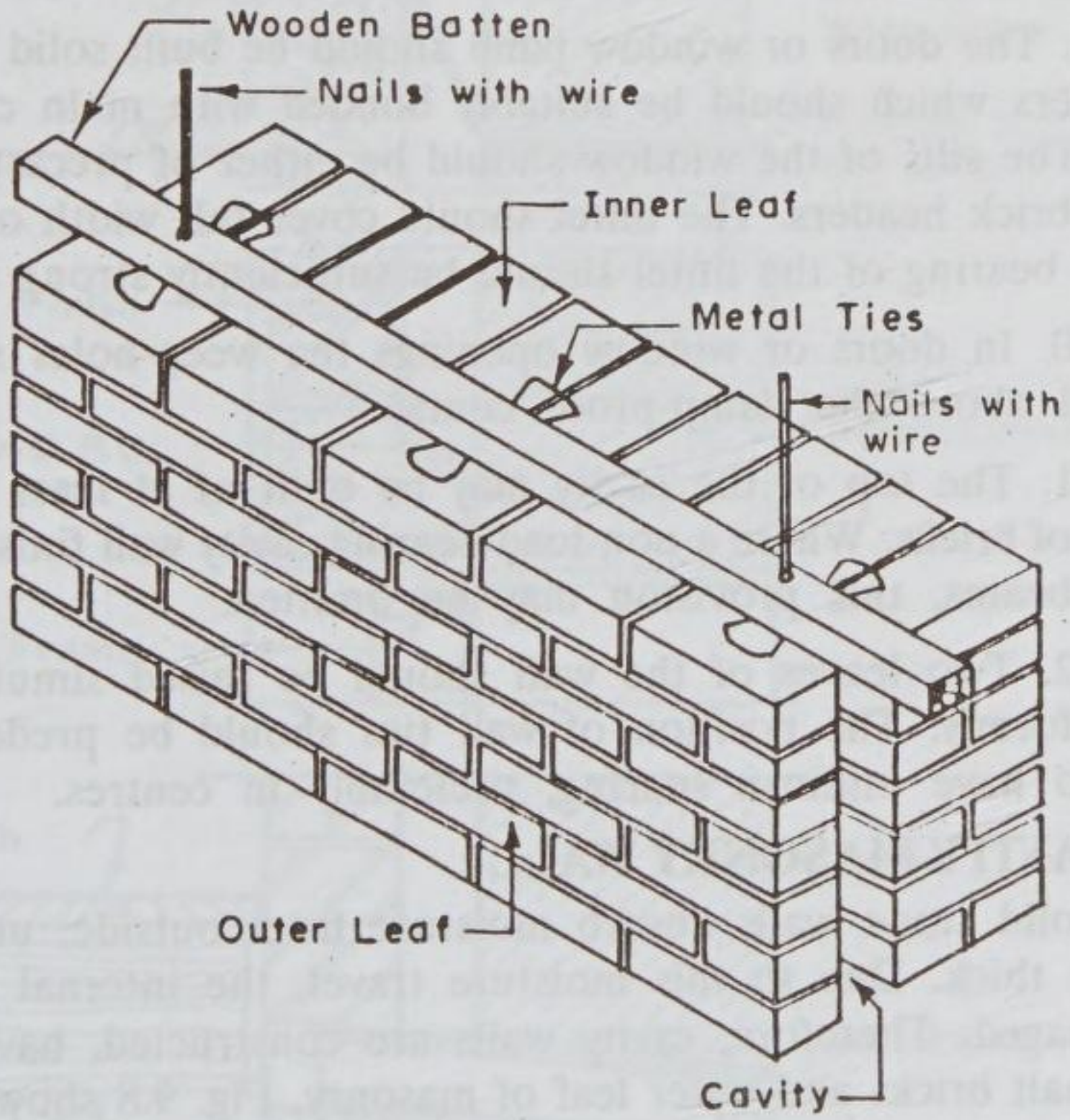


FIG. 9.7. CAVITY WALL CONSTRUCTION.

Summary of Precautions.

1. The contact between the inner and outer leaves should be the least.

2. Ties should be strong and rust proof. They should not permit transmission of water along it from outer face to the inner face.

3. The damp proof course should be laid separately for both leaves.

4. Bottom most horizontal damp proof course should be laid

atleast 150 mm above the bottom of cavity, or above the top of concrete fill in the cavity.

5. The bottom of cavity should be well-ventilated by use of air bricks and ducts.

6. Weep hole or narrow vertical joints should be left in the first course about the bottom of cavity (or top of concrete fill in it), at some regular interval, to drain out rain water collected in the cavity, if any.

7. The bottom of the cavity, or the top of concrete fill in it should be kept atleast 150 mm above the ground level.

8. Wall should be constructed with greater care so that mortar droppings or brick rubbish etc. do not fall inside the cavity.

9. The doors or window jamb should be built solid by means of headers which should be suitably bonded with main cavity wall leaves. The sills of the window should be either of precast concrete slab or brick headers. The lintel should cover full width of the wall and the bearing of the lintel should be sufficiently strong and solid.

10. In doors or window openings the weep-holes should be provided above the damp-proof course.

11. The top of the cavity may be built of at least two solid courses of bricks. Where a non-load bearing cavity wall finishes under R.C.C. beams, this provision may be omitted.

12. Two leaves of the wall should be raised simultaneously and uniformly. The position of wall ties should be predetermined so as to have uniform spacing, preferabiy in centres.

9.8. CAVITY MASONRY WALL

Solid stone walls absorb moisture from outside, unless they are very thick. Due to this moisture travel, the internal finishings are damaged. Therefore, cavity walls are constructed, having inner leaf of half bricks and outer leaf of masonry. Fig. 9.8 shows a cavity wall with outer leaf of ashlar. Fig. 9.9 shows another cavity wall with outer leaf of rubble masonry and inner leaf of brick, supporting a sloping roof.

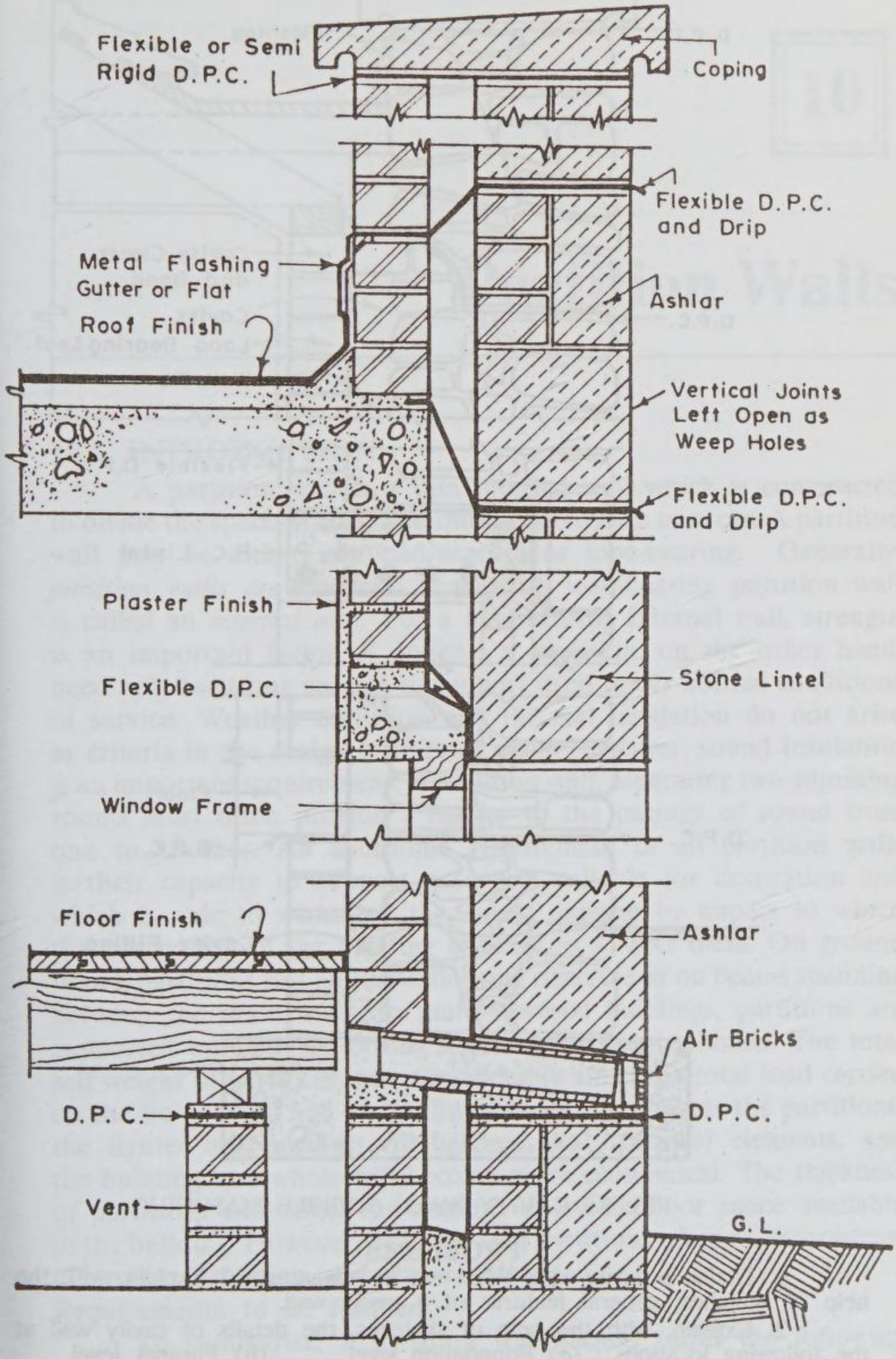


FIG. 9.8. CAVITY WALL (ASHLAR MASONRY).

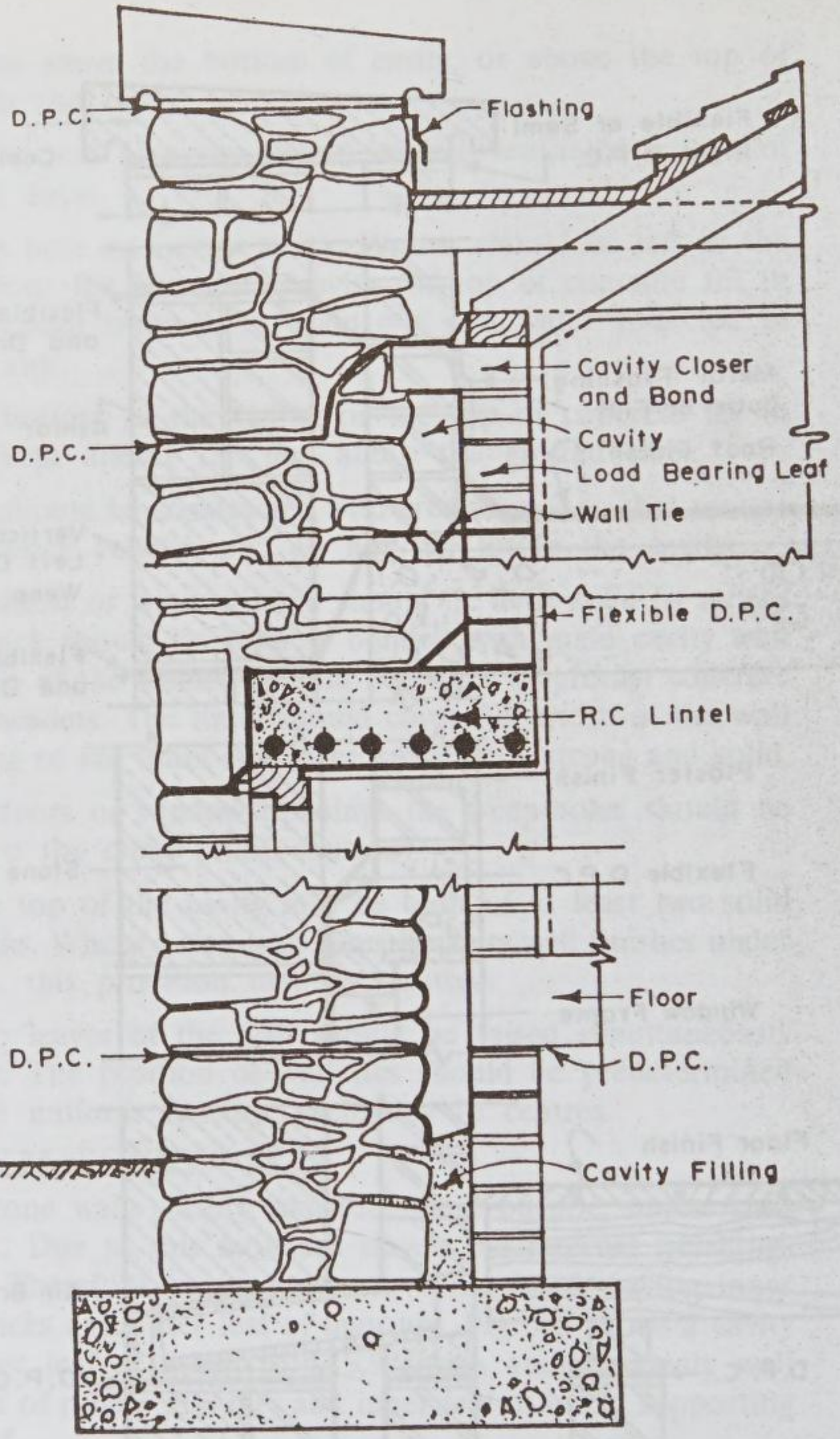


FIG. 9.9 CAVITY WALL (RUBBLE MASONRY).

PROBLEMS

1. Define a cavity wall. What are its advantages ? Explain, with the help of sketches, general features of a cavity wall.

2. Explain, with the help of sketches, the details of cavity wall at the following locations : (*a*) Foundation level (*b*) Parapet level (*c*) Window still level (*d*) Lintel level.

3. Write a note on 'method of construction' of cavity walls. What precautions do you observe in its construction ?

4. Show, with the help of sketches, details of cavity wall in stone masonry.

10

Partition Walls

10.1. INTRODUCTION

A partition wall is a thin internal wall which is constructed to divide the space within the building into rooms or areas. A partition wall may be either non-load-bearing or load-bearing. Generally, *partition walls are non-load bearing.* A load-bearing partition wall is called an *internal wall.* For a load-bearing internal wall, strength is an important factor of design ; a partition, on the other hand, need only be strong enough to support itself under normal conditions of service. Weather exclusion and thermal insulation do not arise as criteria in the design of internal walls. However, sound insulation is an important requirement. A partition wall, separating two adjoining rooms must often provide a barrier to the passage of sound from one to another. An additional requirement in all partition walls is their capacity to support a surface suitable for decoration and which is able to withstand the casual damage by impact to which the occupation of the building is likely to subject them. On ground floors, partitions rest either on flooring concrete or on beams spanning between the main walls. In multi-storeyed buildings, partitions are supported on concrete beams spanning between columns. The total self weight of partitions may considerably affect the total load carried on the frame work and on the foundations. The lighter the partitions, the lighter and smaller will become the structural elements, and the building as a whole will become more economical. The thickness of partitions will affect the amount of usable floor space available in the building. However, light and thin partitions often raise problems of sound insulation and fire resistance.

Requirements to be fulfilled

To summarise, a partition wall should fulfill the following requirements :

1. The partition wall should be strong enough to carry its own load.

2. The partition wall should be strong enough to resist impact to which the occupation of the building is likely to subject them.

3. The partition wall should have the capacity to support suitable decorative surface.

4. A partition wall should be stable and strong enough to support some wall fixtures, wash-basins etc.

5. A partition wall should be as light as possible.

6. A partition wall should be as thin as possible.

7. A partition wall should act as a sound barrier, specially when it divides two rooms.

8. A partition wall should be fire resistant.

Types of partition walls

Partition walls are of the following types :

1. Brick partitions.
2. Clay block partitions.
3. Concrete partitions.
4. Glass partitions.
5. Metal lath partitions.
6. Asbestos sheet or G.I. sheet partitions.
7. Plaster slab partition.
8. Wood-wood slab partition.
9. Timber partitions

10.2. BRICK PARTITIONS

Brick partitions are quite common since they are the cheapest. Brick partitions are of three types :

1. Plain brick partitions
2. Reinforced brick partitions.
3. Brick nogging partitions.

Plain Brick Partitions

Plain brick partitions are usually half brick thick. The bricks are laid as stretchers, in cement mortar. Vertical joints are staggered alternate blocks. The wall is plastered on both the sides. The wall is considerably strong and fire resistant.

Reinforced Brick Partitions

These are stronger than the ordinary brick partitions, and is used when better longitudinal bond is required, and when the partition wall has to carry other super-imposed loads. The thickness of the wall is kept equal to half brick (10 cm). The reinforcement consists of steel meshed strips, called *Exmet,* made from thin rolled steel plates which are cut and stretched (or expanded) by a machine to a diamond network. Such a strip is known as *expanded metal* and is provided at every third course. Another form of meshed rein-

forcement, called *Bricktor* is made of a number of straight tension wires with binding wires (Fig. 7.5 *b*).

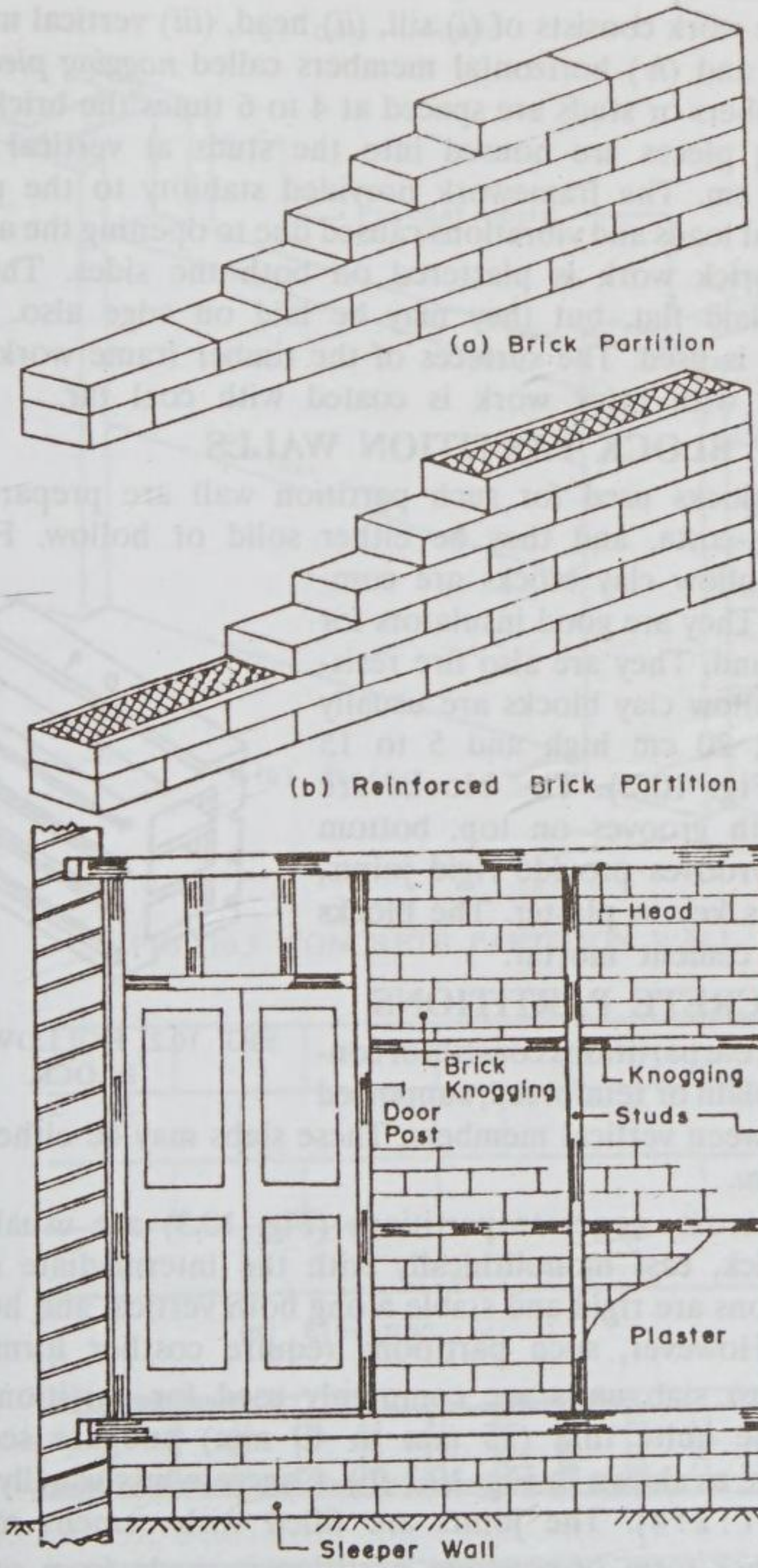


FIG. 10.1. BRICK PARTITION WALLS.

Brick Nogging Partitions

Brick nogging partition wall consists of brick work (half brick thickness) built up within the frame work of wooden members. The timber frame work consists of (*i*) sill, (*ii*) head, (*iii*) vertical members, called *studs,* and (*iv*) horizontal members called *nogging pieces.* The vertical members or studs are spaced at 4 to 6 times the brick length. The nogging pieces are housed into the studs at vertical interval of 60 to 90 cm. The framework provided stability to the partition against lateral loads and vibrations caused due to opening the adjoining door. The brick work is plastered on both the sides. The bricks are usually laid flat, but they may be laid on edge also. Cement mortar, 1 : 3 is used. The surfaces of the timber frame work coming into contact with brick work is coated with coal tar.

10.3. CLAY BLOCK PARTITION WALLS

The blocks used for such partition wall are prepared from clay or terra-cotta, and they be either solid or hollow. For light partitions, hollow clay blocks are commonly used. They are good insulators for heat and sound. They are also fire resistant. The hollow clay blocks are usually 30 cm long, 20 cm high and 5 to 15 cm wide (Fig. 10.2). The blocks are provided with grooves on top, bottom and sides. Grooves provide rigid joints, and serves as key to plaster. The blocks are laid in cement mortar.

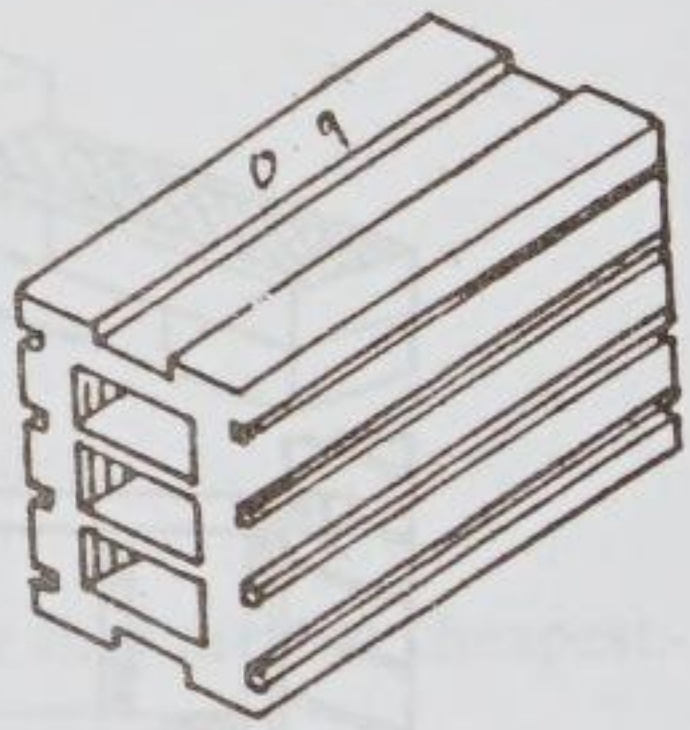


FIG. 10.2. HOLLOW CLAY BLOCK.

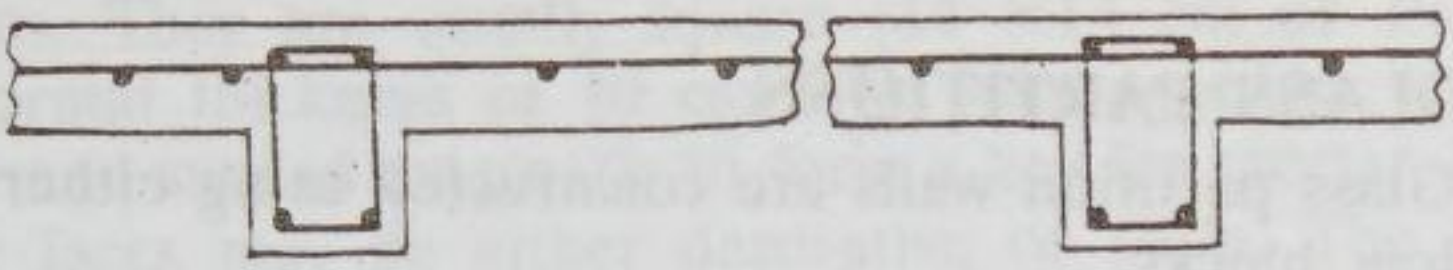
10.4. CONCRETE PARTITIONS

Concrete partitions consists of concrete slabs, plain or reinforced, supported laterally between vertical members. These slabs may be either *precast or cast-in-situ.*

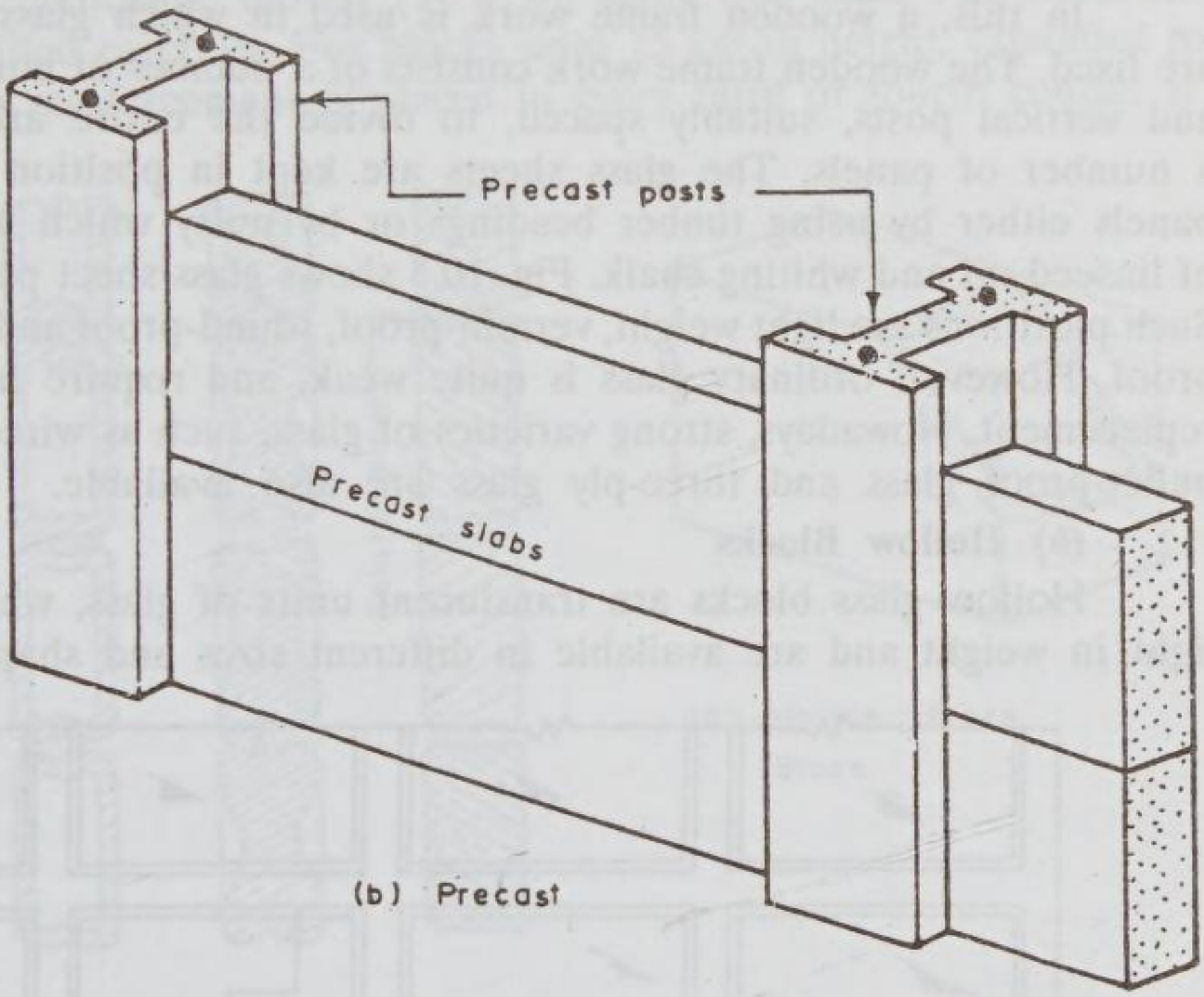
.*Cast-in-situ* concrete partitions (Fig. 10.3) are usually 80 to 100 mm thick, cast monolithically with the intermediate columns. Such partitions are rigid and stable along both vertical and horizontal directions. However, such partitions require costlier form work.

Pre-cast slab units are commonly used for partitions. These slabs may be quite thin (25 mm to 40 mm) and are secured to precast posts, as shown in Fig. 10.3 (*b*). Concrete mix usually adopted is M 150 (1 : 2 : 4). The joints are filled with cement mortar.

Another form of concrete partition is made from precast T-shaped or L-shaped units, as shown in Fig. 10.4. A light weight, hollow partition is obtained, without any necessity of vertical post etc. Cement mortar (1 : 3) is used for jointing.



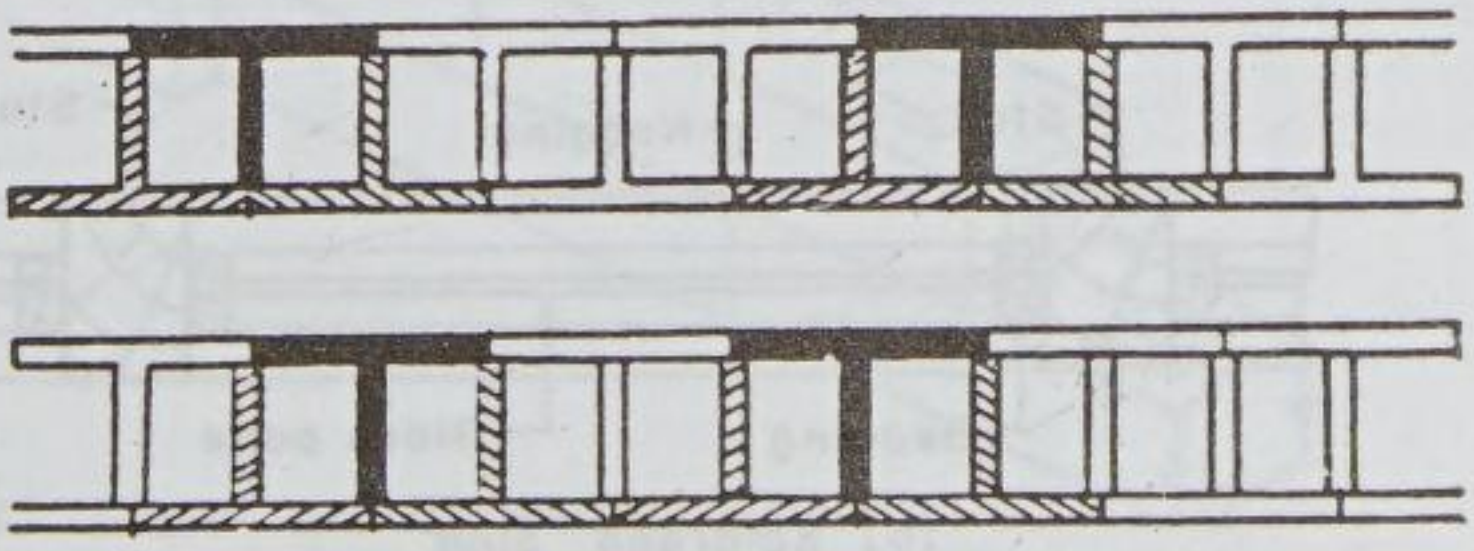
(a) Cast – in – situ



(b) Precast

FIG. 10.3. CONCRETE PARTITION WALL.

(a) Elevation



(b) Plan of Alternate Courses

FIG. 10.4. PRECAST CONCRETE UNITS.

10.5 . GLASS PARTITIONS

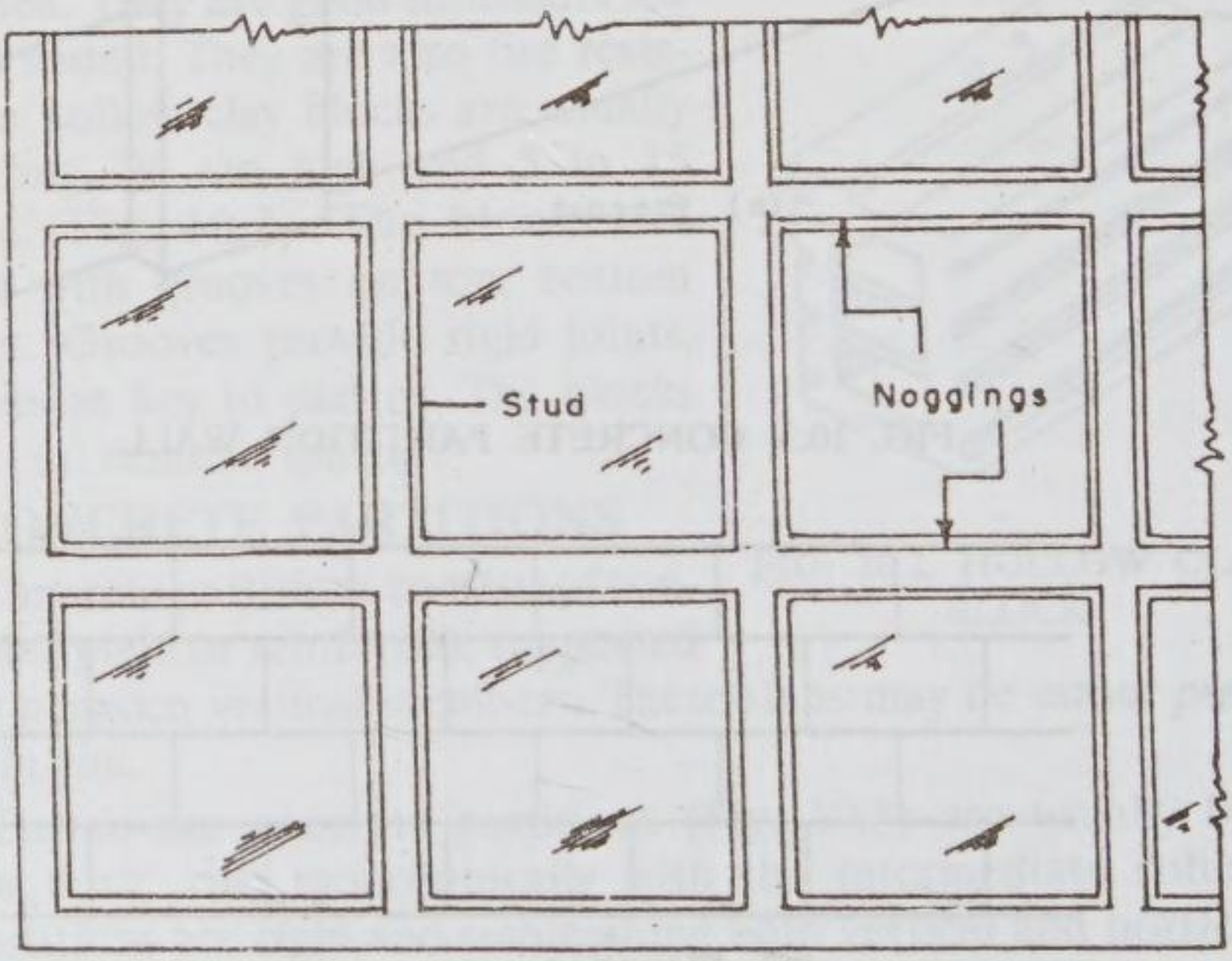
Glass partition walls are constructed using either *glass sheets* or hollow blocks.

(*a*) Glass sheet partition

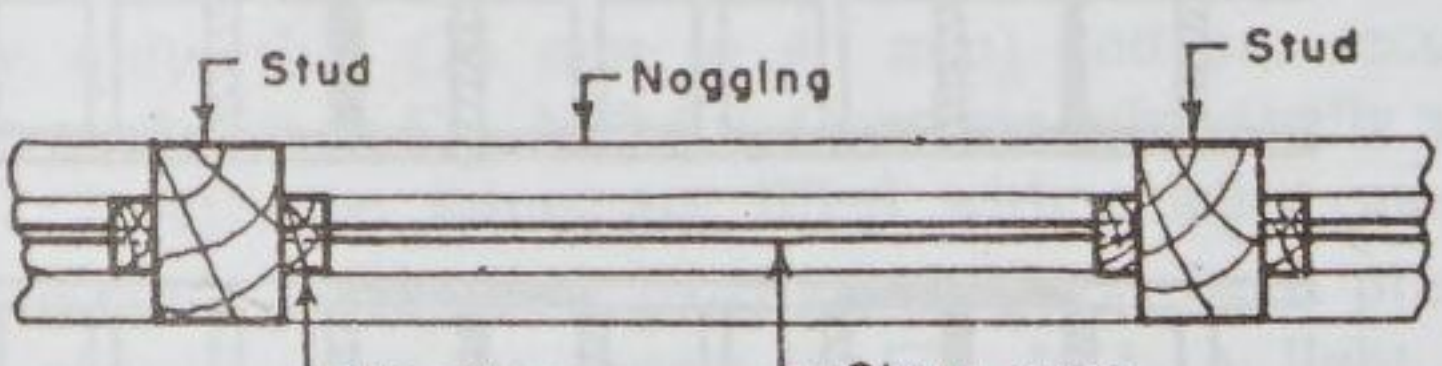
In this, a wooden frame work is used in which glass sheets are fixed. The wooden frame work consists of a number of horizontal and vertical posts, suitably spaced, to divide the entire area into a number of panels. The glass sheets are kept in position in the panels either by using timber beadings or by putty which is made of linseed oil and whiting chalk. Fig. 10.5 shows glass sheet partition. Such partitions are light weight, vermin-proof, sound-proof and damp-proof. However, ordinary glass is quite weak, and require frequent replacement. Nowadays, strong varieties of glass, such as wired glass, bullet-proof glass and three-ply glass are also available.

(*b*) Hollow Blocks

Hollow glass blocks are translucent units of glass, which are light in weight and are available in different sizes and shapes and



(a) Elevation



(b) Enlarged plan

FIG. 10.5. GLASS PARTITION.

thicknesses. They are usually square (14 ×14 cm or 19×19 cm), with a normal thickness of 10 cm. The jointing edges are painted internally and sanded externally to form a key for mortar. The front and back faces may be either decorative or plain. The front and back faces are sometimes fluted. The glass blocks are usually laid in cement-lime mortar (1 : 1 : 4), using fine sand. All joints should be filled carefully. For blocks upto 15 cm in height, expanded metal strip reinforcement is placed in every third or fourth course. If the

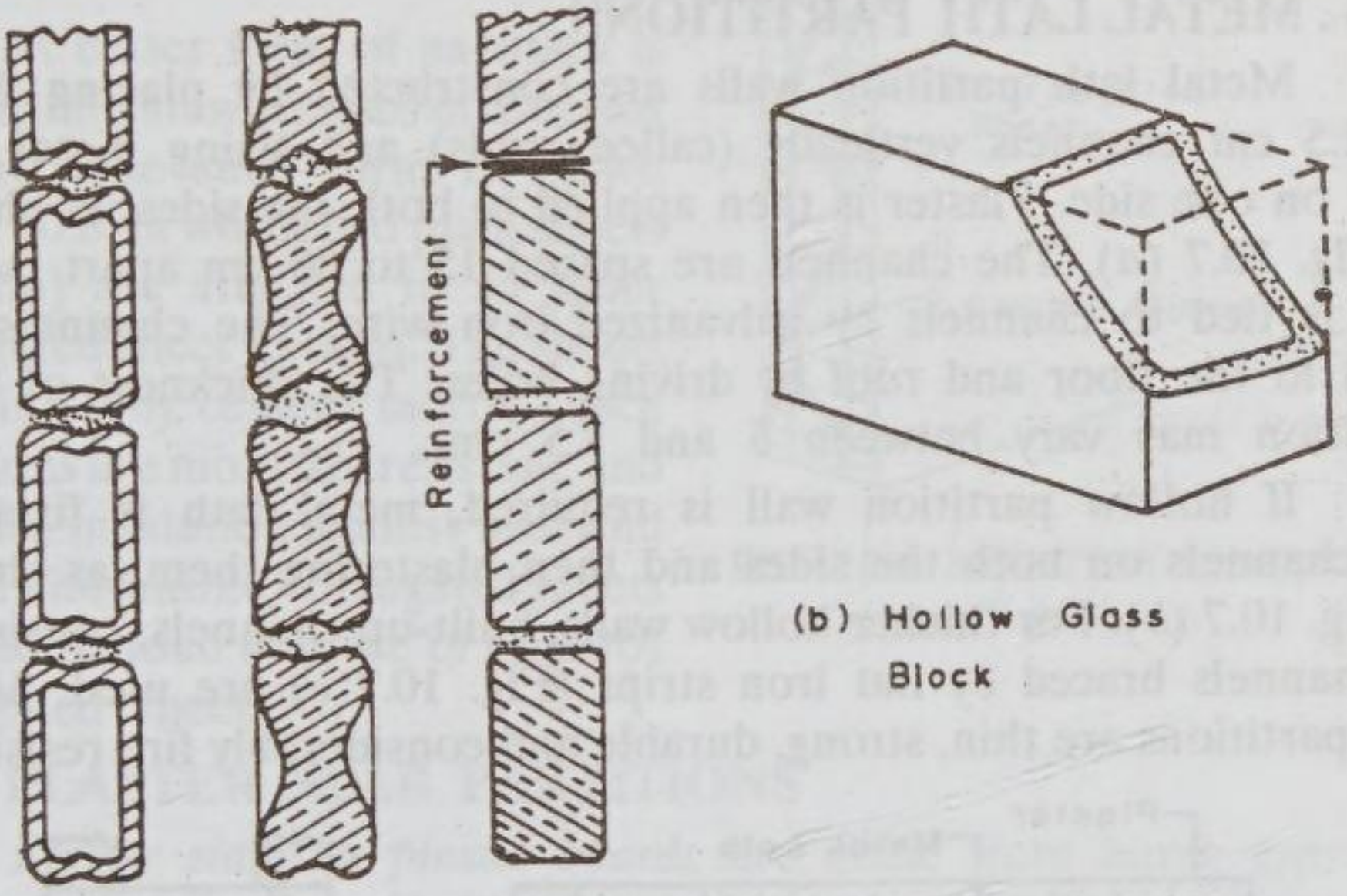


FIG. 10.6. GLASS BLOCK AND GLASS BRICKS WALLS.

height of the block is more than 25 cm, the reinforcement is placed in every course. Provision for expansion should be suitably made along the jambs and head of each panel.

Another type of glass blocks are in the form of *glass bricks* with joggles and end grooves, as shown in Fig. 10.6 (*c*).

Glass blocks or glass bricks walls provide good architectural effect and also admit light. They are sound-proof, fire-proof and heat-proof to some extent.

10.6. METAL LATH PARTITIONS

Metal lath partition walls are constructed by placing 2 cm or 2.5 cm channels vertically (called studs) and fixing metal lath to it on one side. Plaster is then applied to both the sides, as shown in Fig. 10.7 (*a*). The channels are spaced 15 to 30 cm apart. Metal lath is tied to channels by galvanized iron wire. The channels are fixed to the floor and roof by driving holes. The thickness of such partition may vary between 5 and 7.5 cm.

If hollow partition wall is required, metal lath is fixed to the channels on both the sides and then plastering them, as shown in Fig. 10.7 (*b*). For thicker hollow walls, built-up channels, consisting of channels braced by flat iron strips (Fig. 10.7 *c*) are used. Metal lath partitions are thin, strong, durable and considerably fire resistant.

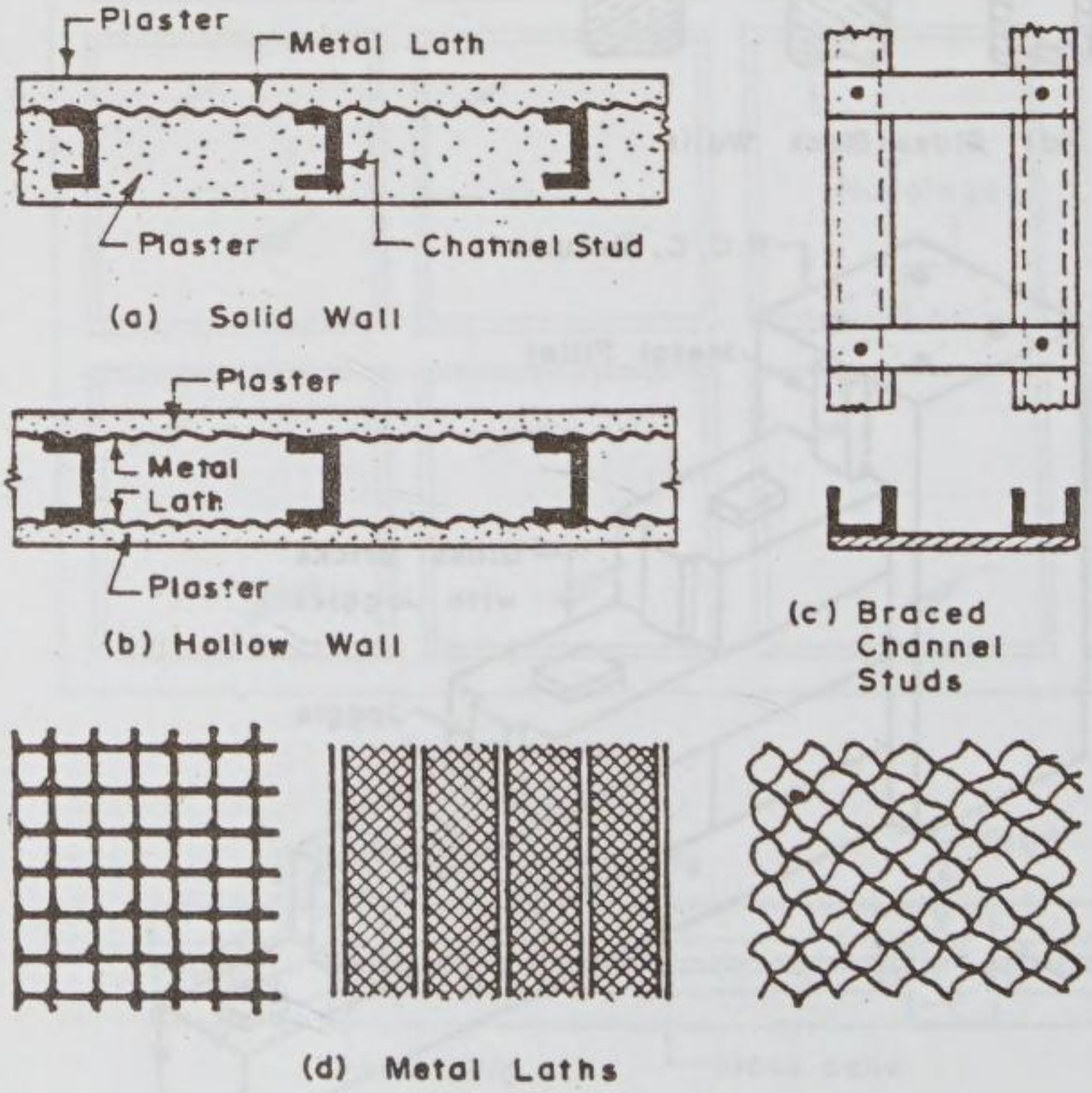


FIG. 10.7. METAL LATH PARTITIONS

10.7. ASBESTOS SHEET OR G.I. SHEET PARTITIONS

Asbestos sheet or G.I. sheets can be fixed to suitable frame of wood, to act as partition wall. The sheets can be fixed either to one side of the frame, or to both the sides. Such partitions are economical, light weight and fairly strong.

A better form of partition is made from patented slabs of asbestos cement. One such form is shown in Fig. 10.8, in which two plain sheets (10 mm) are attached to an inner corrugated sheet (5 mm). The sheets are jointed by cement mortar. Such partitions are more fire resistant, and provides insulation against heat and sound. Galvanized corrugated sheets can also be used in place of asbestos corrugated sheets.

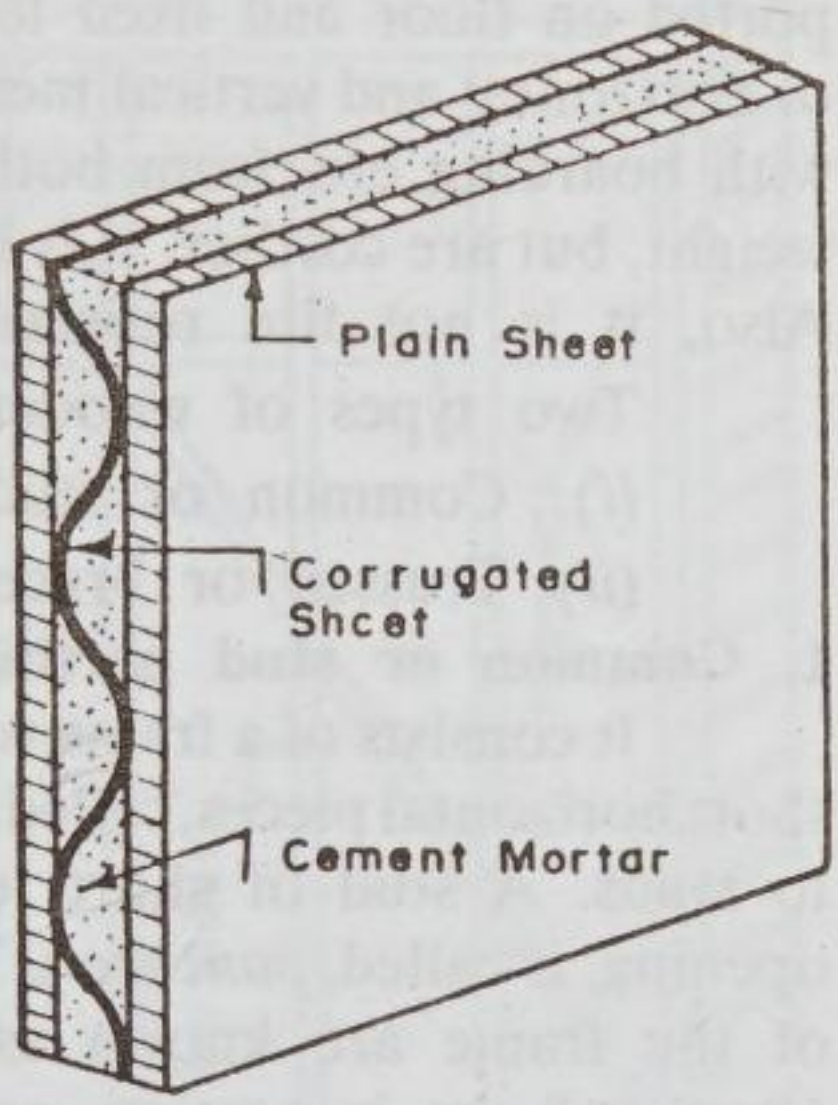


FIG. 10.8. ASBESTOS CEMENT SLABS.

10.8. PLASTER SLAB PARTITIONS

Plaster slabs or plaster boards are made from burnt gypsum or plaster of Paris, mixed with sawdust or other fibrous material to reduce its weight. They are cast in moulds, of size 1 to 2 m long, 30 cm high and 50 to 100 mm thick. Hollow slabs of greater thickness are also cast. Such slabs are light weight and have insulating properties against heat and sound. The surfaces of these slabs may be smooth or rough. Rough surfaces serve as key for plaster. Smooth surfaces are not plastered.

10.9 . WOOD WOOL SLAB PARTITIONS

Wood wool consist of long, tangled, wood fibres, uncompacted, coated and bound together with cement or plaster, and with a rough open surface which provides an excelient key for plaster. Such partitions have sufficient heat and sound insulating properties. They are available in different trade names. The unit weight of such slabs is only 480 kg/m^2; thus such partitions are extremely light weight. Slabs can be sawn and nailed. Vertical mortar joints between the slabs should be staggered. However, wood-wool slabs have a large movement due to changes in moisture content. Such movement must be properly restrained. Care must be taken at the heads of openings to preserve a crack-free plaster finish.

10.10. TIMBER PARTITIONS

Timber partitions consist of wooden frame work, properly supported on floor and fixed to the side walls. This frame work, made of horizontal and vertical members, can either be plastered or covered with boarding etc. from both the sides. Wooden partitions are light weight, but are costlier. It is likely to decay, or eaten away by termites. Also, it is not fire resistant. It's use is reducing day by day.

Two types of wooden partitions may be used :

(*i*) Common or stud partition.

(*ii*) Trussed or braced partition.

1. Common or stud partition

It consists of a frame work of vertical members(called *studs*),and short horizontal pieces, called *noggings*. Horizontal pieces impart rigidity to studs. A stud of short length, such as the one provided on an opening, is called, *puncheon*. The upper and lower horizontal members of the frame are known as *head* and *sill* respectively. The studs, 10 cm × 5 cm in section, are spaced 30 to 45 cm apart. Nogging pieces are cut tightly and fixed between the studs and nailed. The head and sill are 10 cm × 75 mm in section.

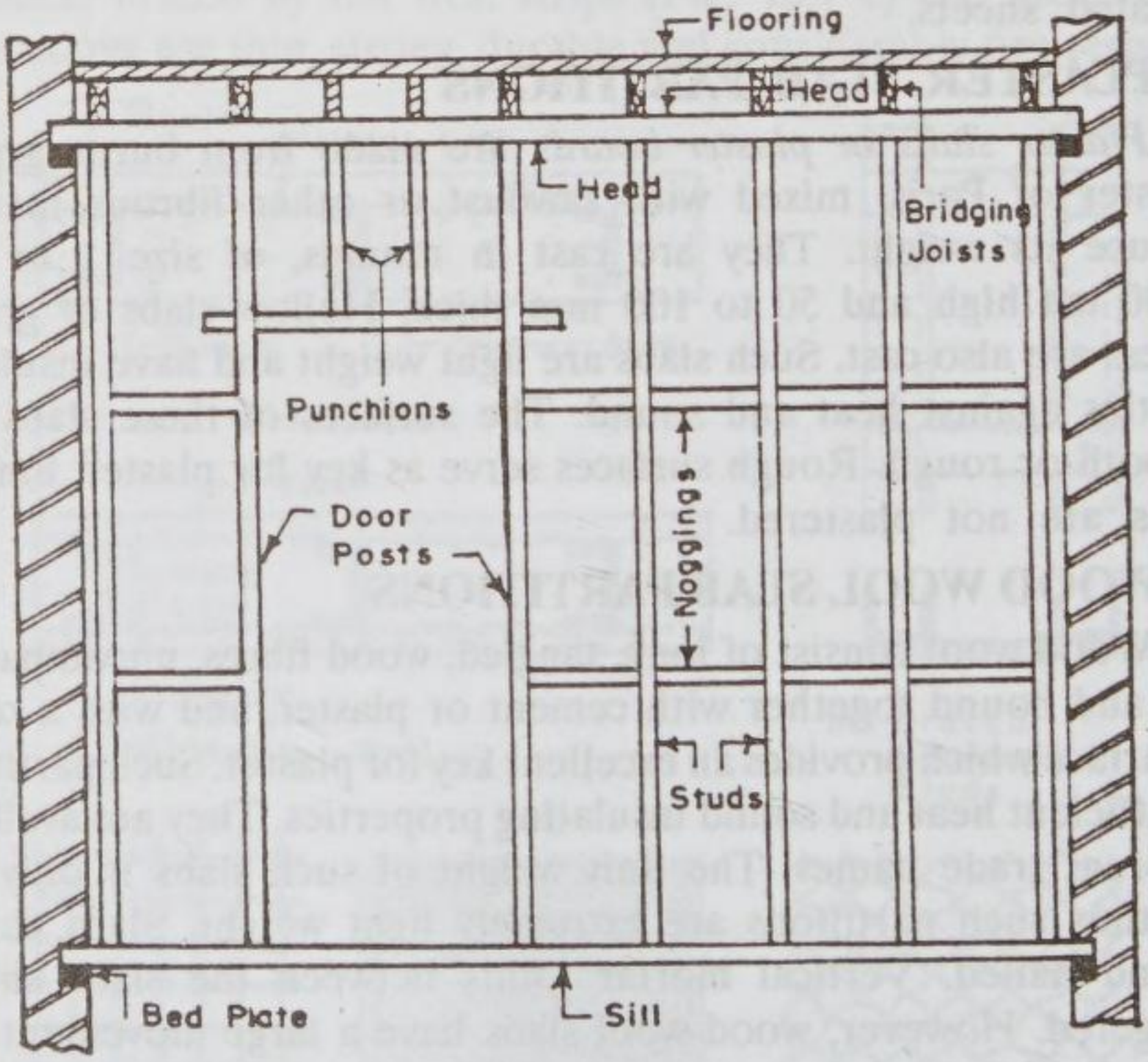
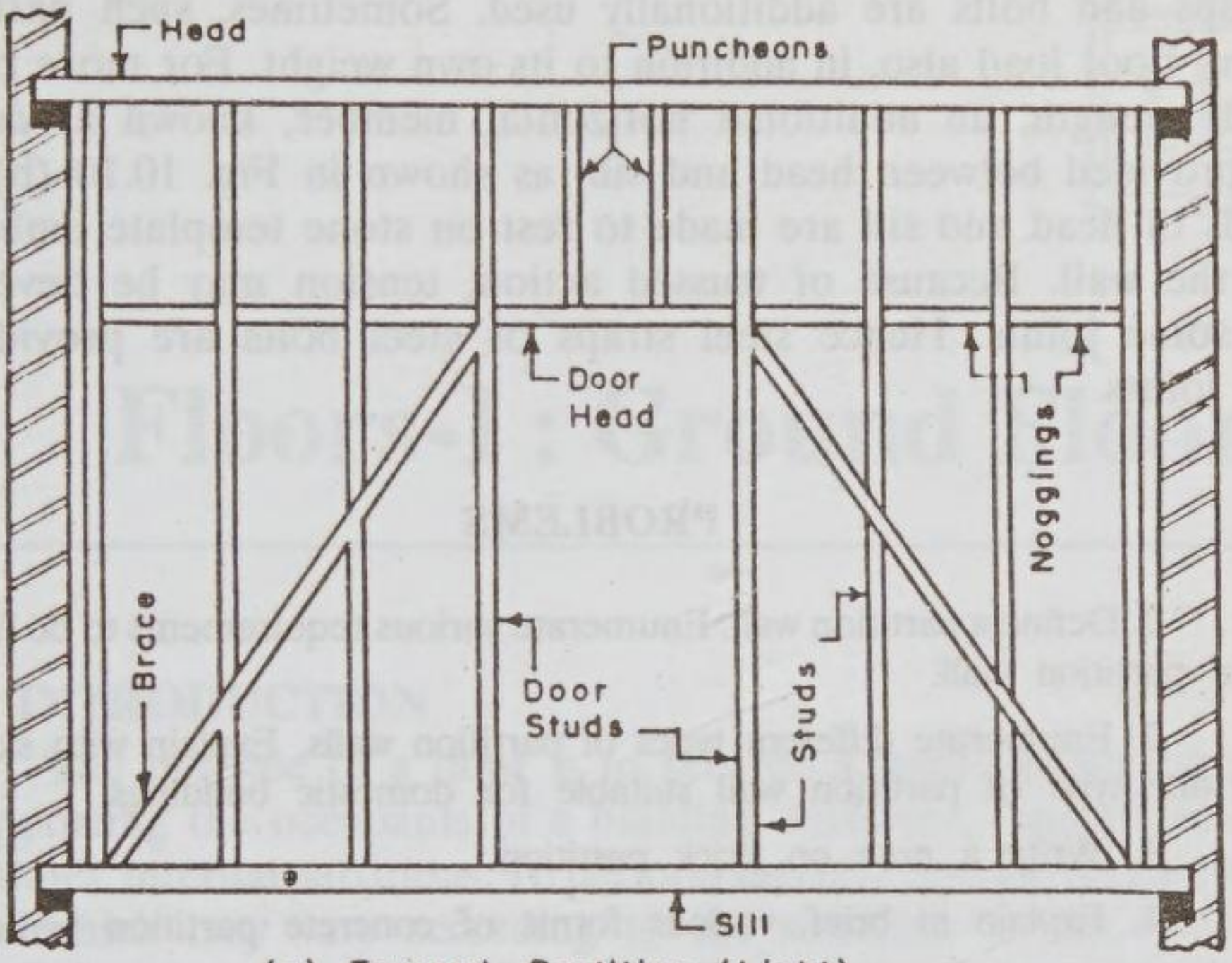


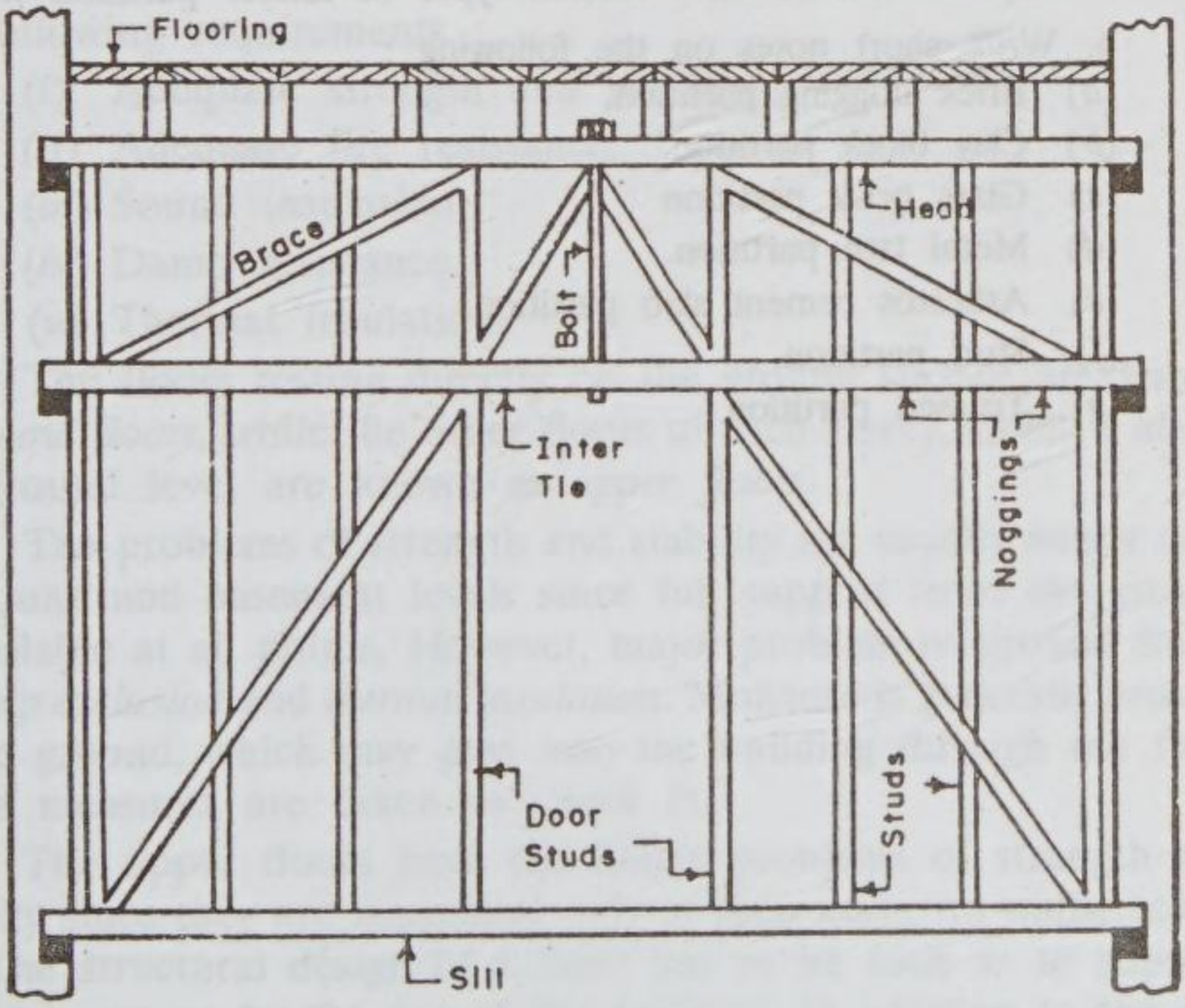
FIG. 10.9. COMMON OR STUD PARTITION.

2. Trussed or braced partitions

Such partitions are provided where there is no means of supporting the partition except at their ends. The frame work is similar



(a) Trussed Partition (Light)



(b) Trussed Partition (Heavy)

FIG. 10.10. TRUSSED PARTITION.

to the stud partition, but inclined members called *braces,* and steel straps and bolts are additionally used. Sometimes, such partitions carry floor load also, in addition to its own weight. For more rigidity and strength, an additional horizontal member, known as *inter-tie* is provided between head and sill, as shown in Fig. 10.10 (b). The ends of head and sill are made to rest on stone template embedded in the wall. Because of trussed action, tension may be developed at some joints. Hence steel straps or steel bolts are provided at *all* joints.

PROBLEMS

1. Define a partition wall. Enumerate various requirements to be fulfilled by a partition wall.

2. Enumerate different types of partition walls. Explain with sketches any one type of partition wall suitable for domestic buildings.

3. Write a note on brick partition.

4. Explain in brief, various forms of concrete partition walls.

5. Explain with sketches various types of timber partition walls.

6. Write short notes on the following :
(*a*) Brick nogging partition.
(*b*) Clay block partition.
(*c*) Glass brick partition
(*d*) Metal lath partition.
(*e*) Asbestos cement slab partition.
(*f*) Stud partition.
(*g*) Trussed partition.

11

Floors-I : Ground Floors

11.1. INTRODUCTION

The purpose of a floor is to provide a level surface capable of supporting the occupants of a building, furniture, equipment and sometimes, internal partitions. To perform this function, and in addition, others which may vary according to the situation of the floor in the building and the nature of the building itself, a floor must satisfy the following requirements :

(*i*) Adequate strength and stability
(*ii*) Adequate fire resistance.
(*iii*) Sound insulation
(*iv*) Damp resistance
and (*v*) Thermal insulation.

The floors resting directly on the ground surface are known as *ground floors,* while the other floors of each storey, situated above the ground level are known as *upper floors.*

The problems of strength and stability are usually minor ones at ground and basement levels since full support from the ground is available at all points. However, major problem of ground floors is *damp exclusion* and *thermal insulation.* Moisture is generally present in the ground, which may pass into the building through the floor unless measures are taken to check it.

The upper floors have the major problems of strength and stability since they are supported only at their ends, on walls, beams etc. The structural design of a floor has to be such as to support the loads set up by the use of the building, in addition to the self weight and the weight of partitions etc. Upper floors do not have problems of damp resistance, though *sound insulation* is generally an important factor in the design. The problem of fire resistance does not arise for the lowest floor of a building, but is often important for upper floors.

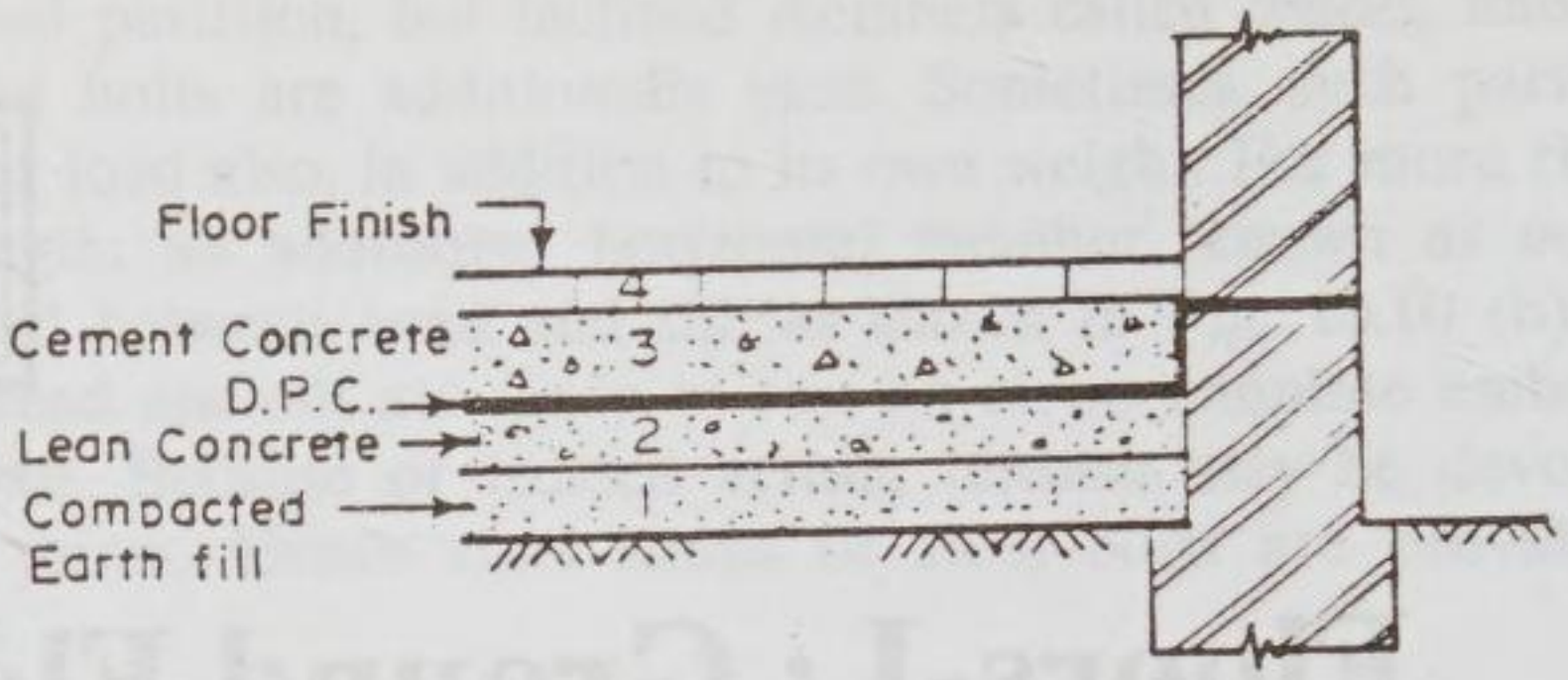


FIG. 11.1. SOLID GROUND FLOOR.

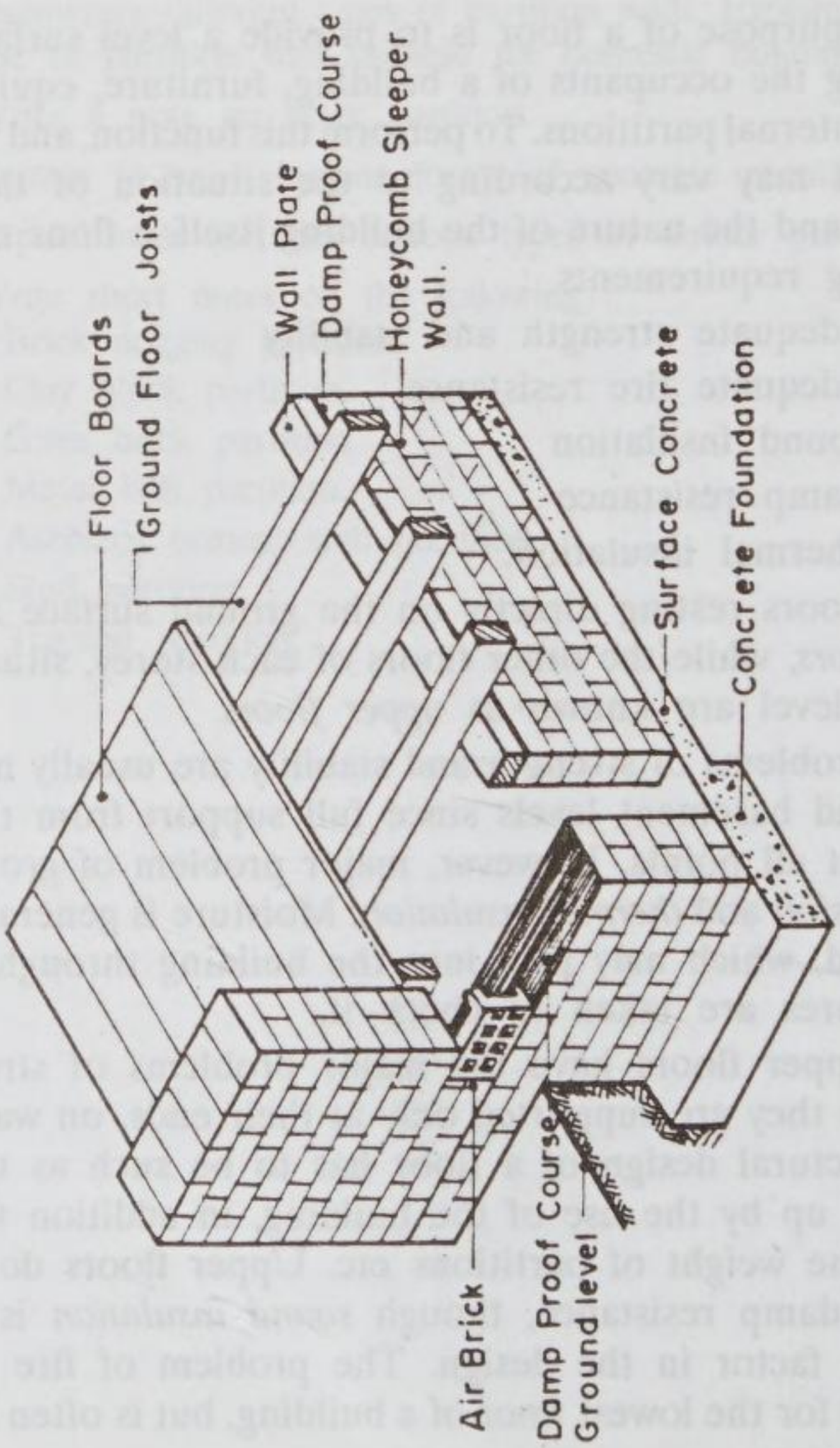


FIG. 11.2. SUSPENDED TIMBER GROUND FLOOR.

11.2. COMPONENTS OF A FLOOR

A floor is composed of two essential components :

(*i*) sub-floor, base course or floor base

(*ii*) Floor covering, or simply, flooring.

The *floor base* is a structural component, which supports the floor covering. For the ground floors, the object of *floor base* is to give proper support to the covering so that it does not settle, and to provide *damp resistance* and *thermal insulation*.

Ground floors may either rest directly on the ground, or may be supported a little distance above the ground. The floors supported directly on the ground are known as *solid floors* (Fig. 11.1) while the floors supported above the ground level are called *suspended floors* (Fig. 11.2). Suspended floors are generally made of timber.

11.3. MATERIALS FOR CONSTRUCTION

Materials used for construction of *ground floor* base are :

(*i*) Cement concrete

(*ii*) Lime concrete

(*iii*) Stones

(*iv*) Bricks

(*v*) Wooden blocks (for wooden flooring only).

The floor base for a solid ground floor is shown in Fig. 11.1. The lowest layer, just above ground surface is that of compacted earth fill. The second layer may either of lean cement concrete or lime concrete or sometimes broken brick bats or stones rammed properly. The third course may be either of cement concrete or of bricks or stones arranged and packed properly. The third layer of cement concrete is more common since it gives proper rigidity to the floor base. Over the third layer of floor base, *floor finish or flooring* is laid.

The materials used for *floor finish* or *floor covering* or flooring are :

1. Mud and Muram
2. Bricks
3. Flag stones
4. Concrete
5. Terrazzo
6. Mosaic
7. Tiles
8. Marble
9. Granolithic finish
10. Wood or timber
11. Asphalt
12. Rubber
13. Linoleum flooring
14. Cork
15. Glass
16. Plastic or P.V.C.

11.4. SELECTION OF FLOORING MATERIAL

Following are the factors that affect the choice of a flooring materials :

1. Initial Cost. The cost of the material should be in conformity

with the type of building, and its likely use. Floor coverings of marble etc. are very costly and may be used only for residential buildings.

2. Appearance. Covering should give pleasing appearance, *i.e.*., it should produce a desired colour effect and architectural beauty. Floorings of terrazzo, mosaic, tiles and marble give good appearance.

3. Cleanliness. The flooring should be capable of being cleaned easily, and it should be non-absorbent. It should have effective resistance against absorption of oil, grease etc.

4. Durability. The flooring should have sufficient resistance to wear, temperature changes, disintegration with time and decay, so that long life is obtained. From this point of view, flooring of marble, terrazzo, tiles, concrete, mosaic etc. are considered to be of best type.

5. Damp resistance. Flooring should offer sufficient resistance against dampness, so that healthy environment is obtained in the building. Flooring of concrete, terrazzo, mosaic etc. are preferred for this purpose, while flooring of cork, wood, rubber, linoleum,, brick etc. are not suitable for damp conditions.

6. Sound Insulation. Flooring should insulate the noise. Also, it should not be such that noise is produced when users walk on it. Cork flooring, rubber flooring and timber flooring are good from this point of view.

7. Thermal Insulation. The flooring should offer reasonably good thermal insulation so that comfort is imparted to the residents of the building. Floor covering of wood, rubber, cork, P.V.C. tiles are better for this purpose.

8. Fire resistance. This is more important for upper floors. Flooring material should offer sufficient fire resistance so that fire barriers are obtained between different levels of a building. Concrete, tiles, terrazzo, mosaic, marble have good fire resistance. Cork, asphalt, rubber and P.V.C. coverings, if used, should be laid on fire resistance base only.

9. Smoothness. The flooring material should be smooth, and should have even surface. However, it should not be slippery.

10. Hardness. It should be sufficiently hard so as to have resistance to indentation marks, imprints etc. likely to be caused by shifting of furniture, equipment etc.

11. Maintenance. The flooring material should require least maintenance. However, whenever repairs are required, it should be such that repairs can be done easily, with least possible expenditure. Hard coverings like tiles, marble, terrazzo, concrete etc. require less maintenance in comparison to materials like cork, wood etc.

11.5. MUD FLOORING AND MURAM FLOORING

Mud and Muram floorings are used only in low cost housing, specially in villages.

Mud flooring. Such flooring is cheap, hard, fairly impervious, easy to construct and easy to maintain. It has good thermal insulation property due to which it remains cool in summer and fairly warm in winter. The method of construction is very easy. Over a well-prepared ground, a 25 cm thick selected moist earth (mostly impervious) is spread and is then rammed well to get a compacted thickness of 15 cm. In order to prevent cracks due to drying, small quantity of chopped straw is mixed in the moist earth, before ramming. Sometimes, cow-dung is mixed with earth and a thin layer of this mix is spread over the compacted layer. Sometimes, a thin paint of cement-cow-dung (1 : 2 to 1 : 3) is applied.

Muram flooring. Muram is a form of disintegrated rock with binding material. This flooring has practically the same properties as that of mud flooring. To construct such a floor, a 15 cm thick layer of muram is laid over prepared subgrade. Over it, a 2.5 cm thick layer of powder muram (fine muram) is spread and water is sprinkled over it. The surface is then rammed well. After ramming, the surface is saturated with a 6 mm thin film of water. The surface is well-trampled under the feet of workmen till the cream of muram rises to the top. The surface is levelled and then kept in that state for a day, and then rammed again with wooden rammers called *thapies* for 3 days, so that dry hard surface is formed. This surface is then smeared or rubbed with thin paste of cow-dung and rammed again for two days, during morning hours. Finally, a coating of mud-cow-dung mix or cement-cow-dung mix is applied over the surface.

11.6. BRICK FLOORING

Such flooring is used in cheap construction, specially where good bricks are available. This flooring is specially suited to ware-

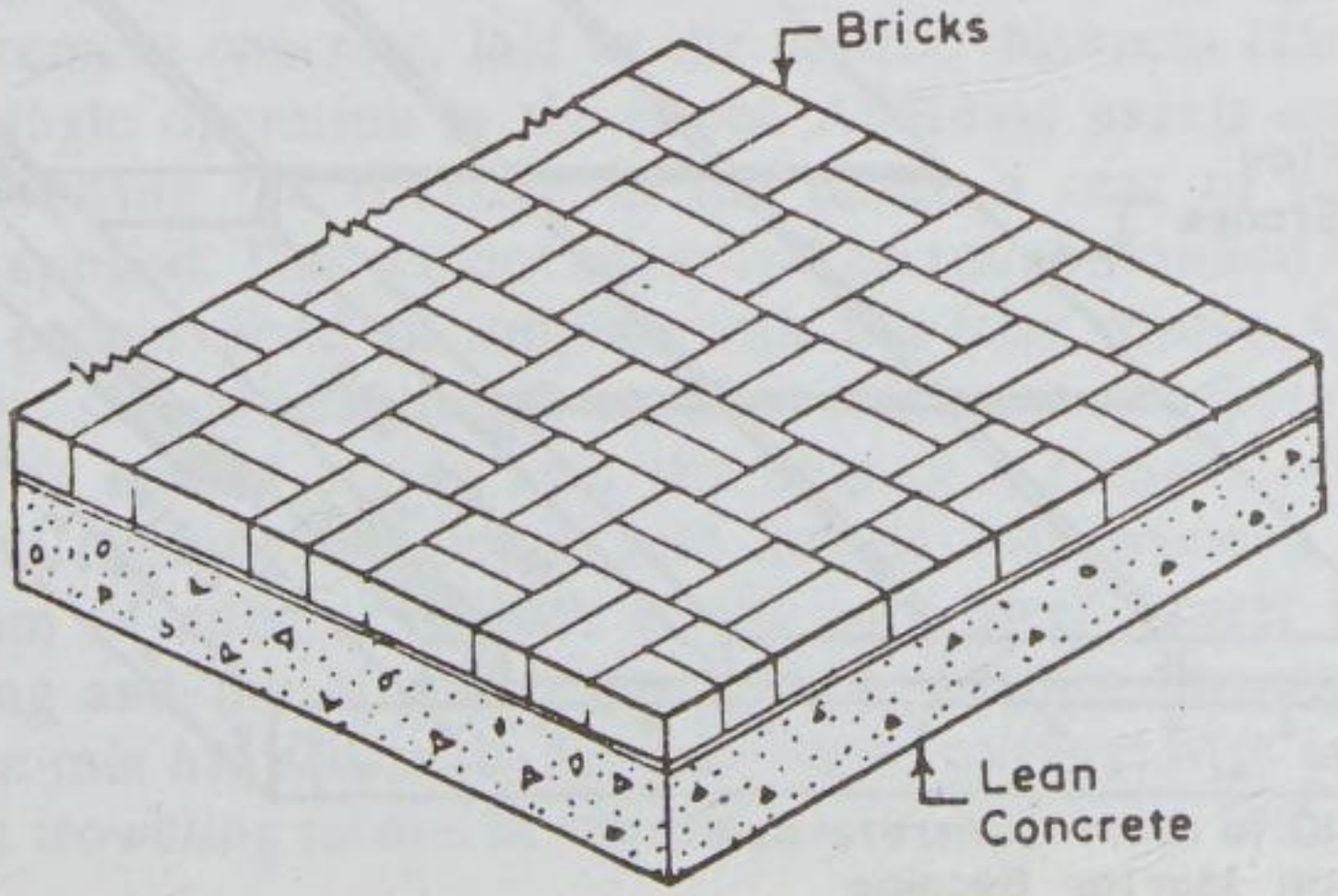


FIG. 11.3. BRICK FLOORING.

houses, stores, godowns etc. Well-burnt bricks of good colour and uniform shapes are used. Bricks are laid either flat or on edge, arranged in herring bone fashion or set at right angles to the walls, or set any other good looking pattern.

The method of preparing the base course for brick flooring varies from place to place. In one method, the subgrade is compacted properly, to the desired level, and a 7.5 cm thick layer of sand is spread. Over this, a course of bricks laid flat in mortar is built. This forms the base course, over which the brick flooring is laid in 12 mm thick bed of cement or lime mortar, in the desired pattern. In the second method, 10 to 15 cm thick layer of lean cement concrete (1 : 8 : 16) or lime concrete is laid over the prepared subgrade. This forms the base course, over which bricks are laid on edge (or flat) on 12 mm thick mortar bed in such a way that all the joints are full with mortar. In both the cases, the joints are rendered flush and finished. The work is then properly cured.

1.17. FLAG STONE FLOORING

Flag stone is any laminated sand stone available in 2 cm to 4 cm thickness, in the form of stone slabs of square (30 cm × 30 cm, 45 cm × 45 cm or 60 cm × 60 cm) or rectangular size (45 × 60 cm). This type of work is also called paving. The stone slabs are laid on concrete base. The sub-soil is properly compacted, over which 10 to 15 cm thick lime concrete or lean cement concrete is laid. This forms the base course of the floor. The flag stones (stone slabs) are then laid over 20 to 25 mm thick layer of bed mortar (Fig. 11.4). In laying the slabs, work is started from two diagonally opposite corners and brought up from both sides. A string is stretched between two corner slabs laid first to correct level. Other slabs are then so laid that their tops touch the string. If any particular slab

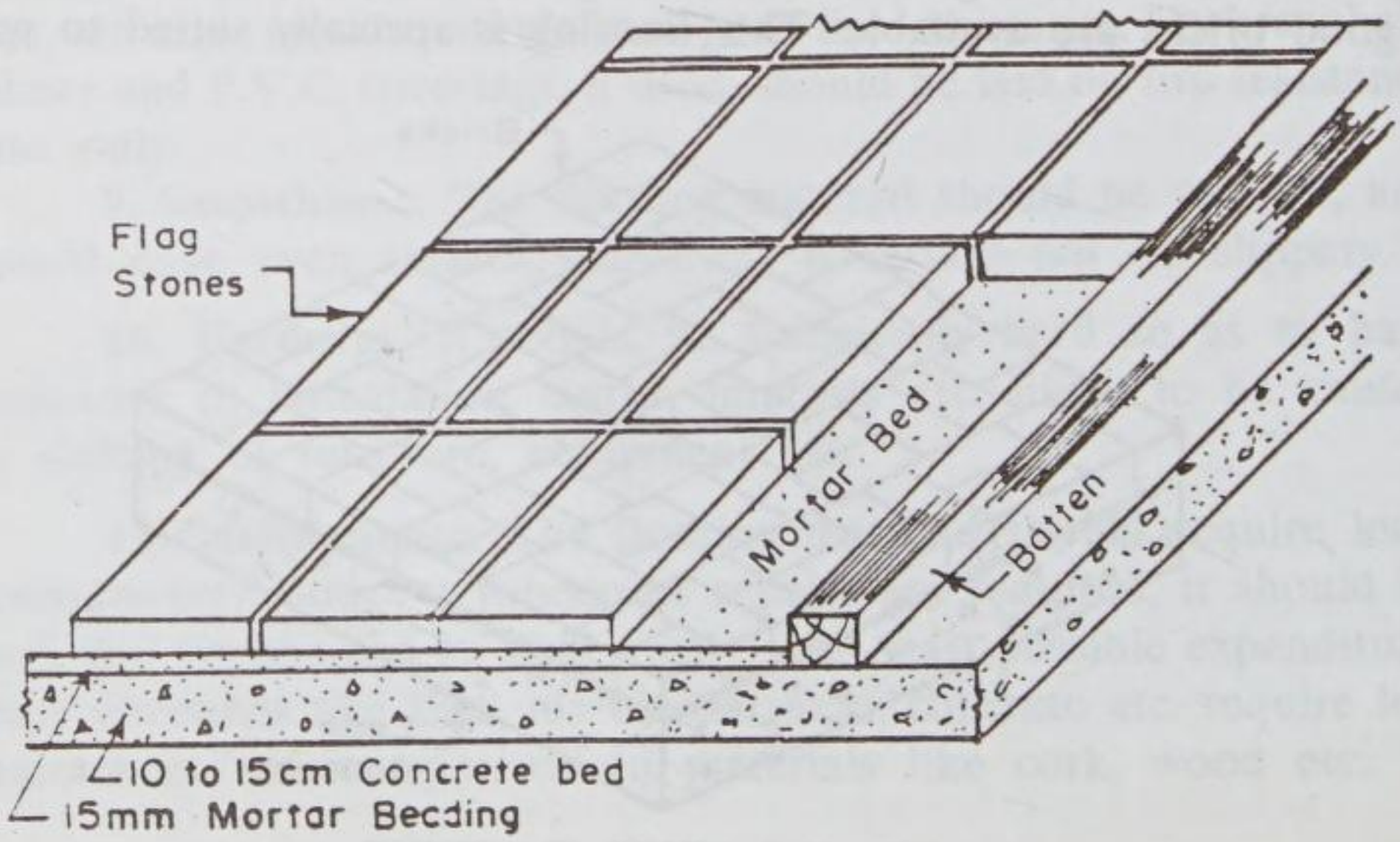


FIG. 11.4. FLAG STONE FLOORING.

falls lower than the string level, it is re-laid by putting fresh layer of stiff mortar. When the stone slabs are properly set, mortar in the joints is raked out to a depth of about 15 to 20 mm and then flush pointed with 1 : 3 cement mortar. Proper slope is given to the surface for drainage. The work is properly cured.

11.8. CEMENT CONCRETE FLOORING

This is commonly used for residential, commercial and even industrial building, since it is moderately cheap, quite durable and easy to construct. The floor consists of two components: (*i*) base concrete, and (*ii*) topping or wearing surface. The two components of the floor can be constructed either monolithically (*i.e.* topping laid immediately after the base course is laid) or non-monolithically. When the floor is laid monolithically, good bond between the two components is obtained resulting in smaller over all thickness. However, such a construction has three disadvantages : (*i*) the topping is damaged during subsequent operations, (*ii*) hair cracks are developed because of the settlement of freshly laid base course which has not set, and (*iii*) work progress is slow because the workman has to wait atleast till the initial setting of the base course. Hence in most of the cases, non-monolithic construction is preferred.

The base course may be 7.5 to 10 cm thick, either in lean cement concrete (1 : 3 : 6 to 1 : 5 : 10) or lime concrete containing 40% mortar of 1 : 2 lime-sand (or 1 lime : 1 surkhi : 1 sand) and 60% coarse aggregate of 40 mm nominal size. The base course is laid over well-compacted soil, compacted properly and levelled to rough surface. It is properly cured.

When the base concrete has hardened, its surface is brushed with stiff broom and cleaned thoroughly. It is wetted the previous night and excess water is grained. The topping is then laid in square or rectangular panels, by use of either glass or plain asbestos strips or by use of wooden battens set on mortar bed. The panels may be 1 × 1 m, 2 × 2 m or 1 × 2 m in size. The topping consists of 1 : 2 : 4 cement concrete, laid to the desired thickness (usually 4 cm) in one single operation in the panel. Alternate panels are laid first. Prior to laying the concrete in the panel, a coat of neat cement slurry is applied. This cement slurry laid on rough-finished base course ensures proper bond of topping with the base course. Glass strips or battens should have depth equal to thickness of topping. Topping concrete is spread evenly with the help of a straight edge, and its surface is thoroughly tamped and floated with wooden floates till the cream of concrete comes at the top. Steel trowel is used for something and finishing the top surface. Further troweling is done when the mix has stiffened. Dusting of the surface with neat cement and then troweling results in smooth finish at the top. Other alternate

layers are then laid after 72 hours, so that initial shrinkage of already laid panels take place, thus eliminating the cracks. The prepared surface is protected from sunlight, rain, other damages for 12 to 20 hours. The surface is then properly cured for a period of 7 to 14 days.

When monolithic construction is laid, the topping is laid 1 hour to 4 hours after placing the base concrete.

Granolithic finish. In industrial building, hard wearing surface is sometimes required. This can be achieved by applying granolithic finish over the concrete topping described above. Granolithic finish consists of rich concrete made with very hard and tough quality coarse aggregate (such as granite, basalt, quartzite etc.) graded from 13 mm to 240 No. I.S. sieve. The concrete mix proportion varies from 1 : 1 : 2 to 1 : 1 : 3 for heavy duty floors to 1 : 2 : 3 for public buildings. The tnickness of finish may be minimum 25 mm when laid monolithically with the top concrete, and 35 mm when laid over hardened surface. However, for public buildings such as schools hospitals etc. the thickness of the finish may be 13 mm to 20 mm. using small size aggregate. If exceptionally hard surface is required, sand may be replaced by fine aggregate of crushed granite, and/or abrasive grit may be sprinkled uniformly over the surface (@ 1.5 to 2.5 kg/m^2), during floating operation.

11.9. TERRAZZO FLOORING

Terrazzo flooring is another type of floor finish that is laid in thin layer over concrete topping. It is very decorative and has good wearing properties. Due to this, it is widely used in residential buildings, hospitals, offices, schools and other public buildings. *Terrazzo* is a specially prepared concrete surface containing cement (white or grey) and marble chips (of different colours), in proportion to $1 : 1\frac{1}{4}$ to 1 : 2. when the surface has set, the chips are exposed by grinding operation. Marble chips may vary from 3 mm to 6 mm size. Colour can be mixed to white cement to set desired tint. The flooring is, however, more expensive.

The sub-base preparation and concrete base laying is done in a similar manner, as explained for cement concrete flooring. The top layer may have about 40 mm thickness, consisting of (*i*) 34 mm thick cement concrete layer (1 : 2 : 4) laid over the base concrete, and (*ii*) about 6 mm thick terrazzo topping.

Before laying the flooring, the entire area is divided into suitable panels of predetermined size and shape. For this, aluminum or glass strips are used. The strips have the same height as the thickness of the flooring (*i.e.* 40 mm). The strips are jointed to the base concrete, with the help of cement mortar, and their tops are perfectly

set to level and line. Alternate panels are filled. The width of the strips may be 1.5 to 2.0 mm.

The surface of base concrete is cleaned of dirt etc. and thoroughly wetted. The wet surface of the base concrete is smeared with cement slurry. Concrete of grade 1 : 2 : 4 is then laid in alternate panels levelled and finished to rough surface. When the surface is hardened, the terrazzo mix (containing cement, marble chips and water) is laid and finished to the level surface. Additional marble chips may be added during tamping and rolling operation, so that atleast 80% of the finished surface show exposed marble chips. The surface is then floated and trowelled, and left to dry for 12 to 20 hours. After that, the surface is cured properly for 2-3 days.

The first grinding is done, preferably by machine, using coarse grade (No. 60) carborundum stones, using plenty of water. The ground surface is then scrabbed and cleaned. Cement grout of cream-like consistency, of the same colour, is then applied on the surface so that pores and holes etc. are filled. The surface is cured for 7 days and then second grinding is done with carborundum stones of fine grade (NO. 120). The surface is scrabbed and cleaned thoroughly, and cement grout is again applied. The surface is cured for 4 to 6 days and final grinding is done with carborundum stones of 320 grit size. The surface is thoroughly scrabbed and cleaned, using plenty of water. The floor is then washed with dilute oxalic acid solution. Finally, the floor is polished, with polishing machines the wheels of which are fitted with felt or hessian bobs, to get fine shine. Wax polish is also applied with the help of the polishing machine, to get final glossy surface.

11.10 MOSAIC FLOORING

Mosaic flooring is made of small pieces of broken tiles of china glazed or of cement, or of marble, arranged in different pattern. These pieces are cut to desired shapes and sizes. A concrete base is prepared as in the case of concrete flooring, and over it 5 to 8 cm thick lime-surkhi mortar is spread and leveled, over an area which can be completed conveniently within working period so that the mortar may not get dried before the floor is finished. On this. a 3 mm thick cementing material, in the form of paste of two parts of slaked lime, one part of powdered marble and one part of puzzolana material, is spread and is left to dry for about 4 hours. Thereafter, small pieces of broken tiles or marble pieces of different colours are arranged in definite patterns and hammered into the cementing layer. The surface is gently rolled by a stone roller of 30 cm dia. and 40 to 60 cm long, sprinkling water over the surface, so that cementing material comes up through the joints, and an even surface is obtained. The surface is allowed to dry for 1 day, and is, thereafter,

rubbed with a pumice stone fitted with a long wooden handle, to get smooth and polish surface. The floor is allowed to dry for two weeks before use.

11.11. TILED FLOORING.

Tiled flooring is constructed from square, hexagonal or other shapes, made of clay (pottery), cement concrete or terrazzo. These are available in different sizes and thicknesses. These are commonly used in residential houses, offices, schools, hospitals and other public buildings, as an alternative to *terrazzo* flooring, specially where the floor is to be laid quickly. The method of laying tiled flooring is similar to that for flag stone flooring except that greater care is required. Over the concrete base, a 25 to 30 mm thick layer of lime mortar 1 : 3 (1 lime and 3 sand or surkhi) is spread to serve as bedding. This bedding mortar is allowed to harden for 12 to 24 hours. Before laying the tiles, neat cement slurry is spread over the bedding mortar and the tiles are laid flat over it, gently pressing them into the bedding mortar with the help of wooden mallet, till levelled surface is obtained. Before laying the tiles, thin paste of cement is applied on their sides, so that the tiles have a thin coat of cement mortar over the entire perimeter surface. Next day, the joints between adjacent tiles are cleaned of loose mortar etc. to a depth of 5 mm, using wire brush, and then grouted with cement slurry of the same colour shade as that of the tiles. The slurry is also applied over the flooring in thin coat. The flooring is then cured for 7 days, and then grinding and polishing is done in the same manner as that for terrazzo flooring.

11.12. MARBLE FLOORING

It is a superior type of flooring, used in bath-rooms and kitchens of residential buildings, and in hospitals, sanitoriums, temples etc. where extra cleanliness is an essential requirement. Marble slabs may be laid in different sizes, usually in rectangular or square shapes. The base concrete is prepared in the same manner as that for concrete flooring. Over the base concrete, 20 mm thick bedding mortar of either 1 : 4 cement : sand mix or 1 (lime putty) : 1 (surkhi) : 1 coarse sand mix is spread under the area of each individual slab. The marble slab is then laid over it, gently pressed with wooden mallet and levelled. The marble slab is then again lifted up, and fresh mortar is added to the hollows of the bedding mortar. The mortar is allowed to harden slightly, cement slurry is spread over it, the edges of already laid slabs are smeared with cement slurry paste, and then the marble slab in question is placed in position. It is gently pushed with wooden mallet so that cement pastes oozes out from the joint which should be as thin as possible (paper thick). The oozed out cement is cleaned with cloth. The paved area is

properly cured for about a week.

11.13. TIMBER FLOORING

Timber flooring is used for carpentry halls, dancing halls, auditoriums, etc. They are not commonly used in residential buildings in India, because timber flooring is also quite costlier. However, in hilly areas, where timber is cheaply and readily available, and where temperature drop very low, timber flooring is quite common. One the major problems in timber flooring is the damp prevention. This can be done by introducing D.P.C. layer below the flooring.

Timber floors can either be of 'suspended type' (*i.e.*. supported above the ground) or 'solid type' (fully supported on the ground). The suspended type timber flooring is shown in Fig. 11.2. An alternative sketch of 'suspended' or 'supported' timber flooring is shown in Fig. 11.5. The hollow space between the flooring and over site concrete is kept dry and well-ventilated by providing air bricks in the outer walls, and voids in the sleeper wall. The flooring consists of boarding supported on bridging or floor joists of timber, which are nailed to the wall plates at their ends. Sleeper walls are not spaced more than 1.8 to 2 m.

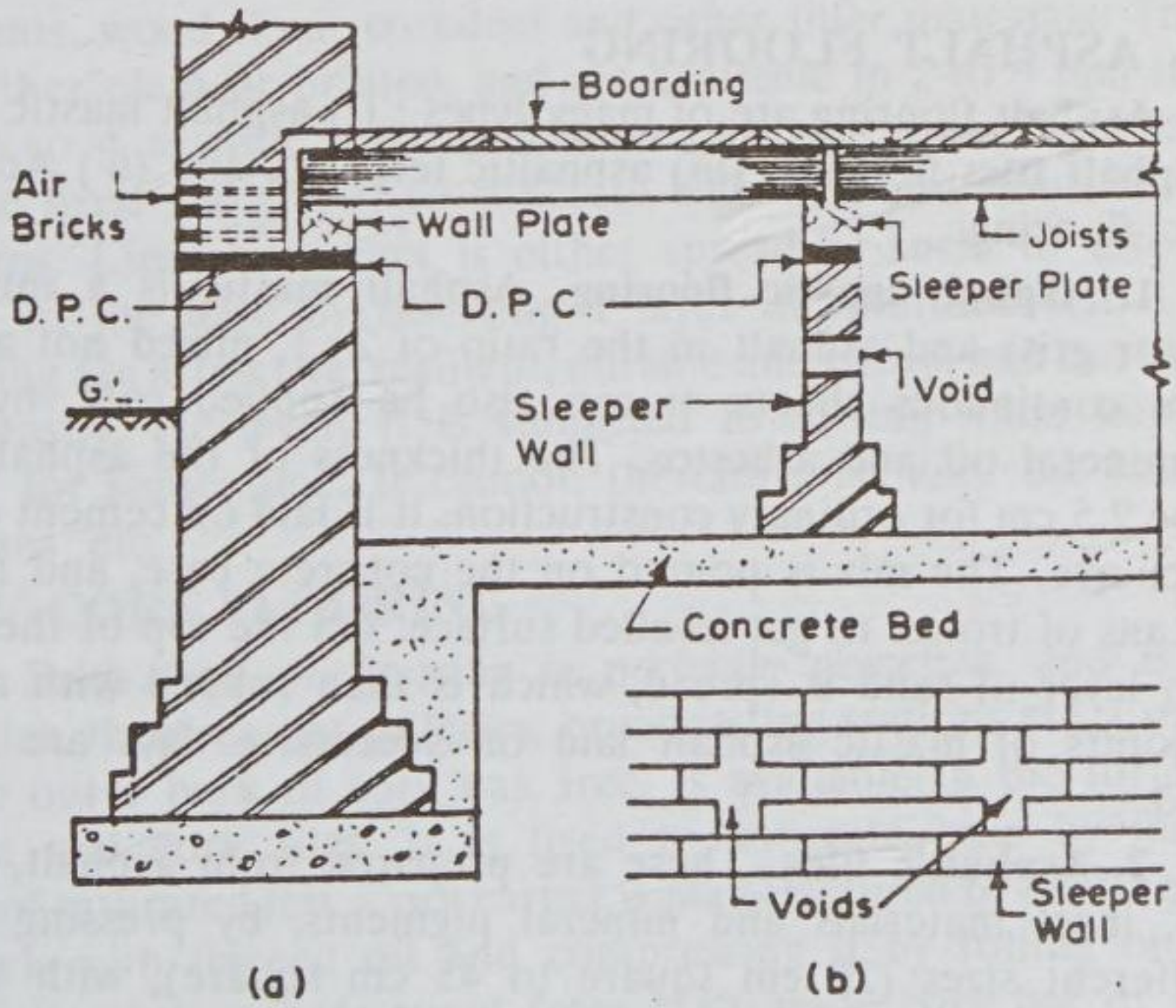


FIG. 11.5. SUPPORTED TYPE TIMBER FLOOR.

Where the problems of dampness is not acute, timber floors may be supported on the ground all along. For this type of construction, base concrete is first laid in 15 to 20 cm thickness. Over it, a layer of mastic asphalt is applied. Wooden block flooring is then laid over it, as shown in Fig. 11.6. Wooden blocks are short but thick (with sizes 20× 8 cm to 30× 8 cm and thickness 2 to 4 cm) and

are laid in suitable designs. In order to fix the wooden floor on concrete slabs, longitudinal nailing strips, with bevelled section, are embedded in concrete at suitable interval. Sometimes, special concrete, called nailing concrete may be used as an alternative to the nailing strips. Special flooring nails are used for nailing down the flooring.

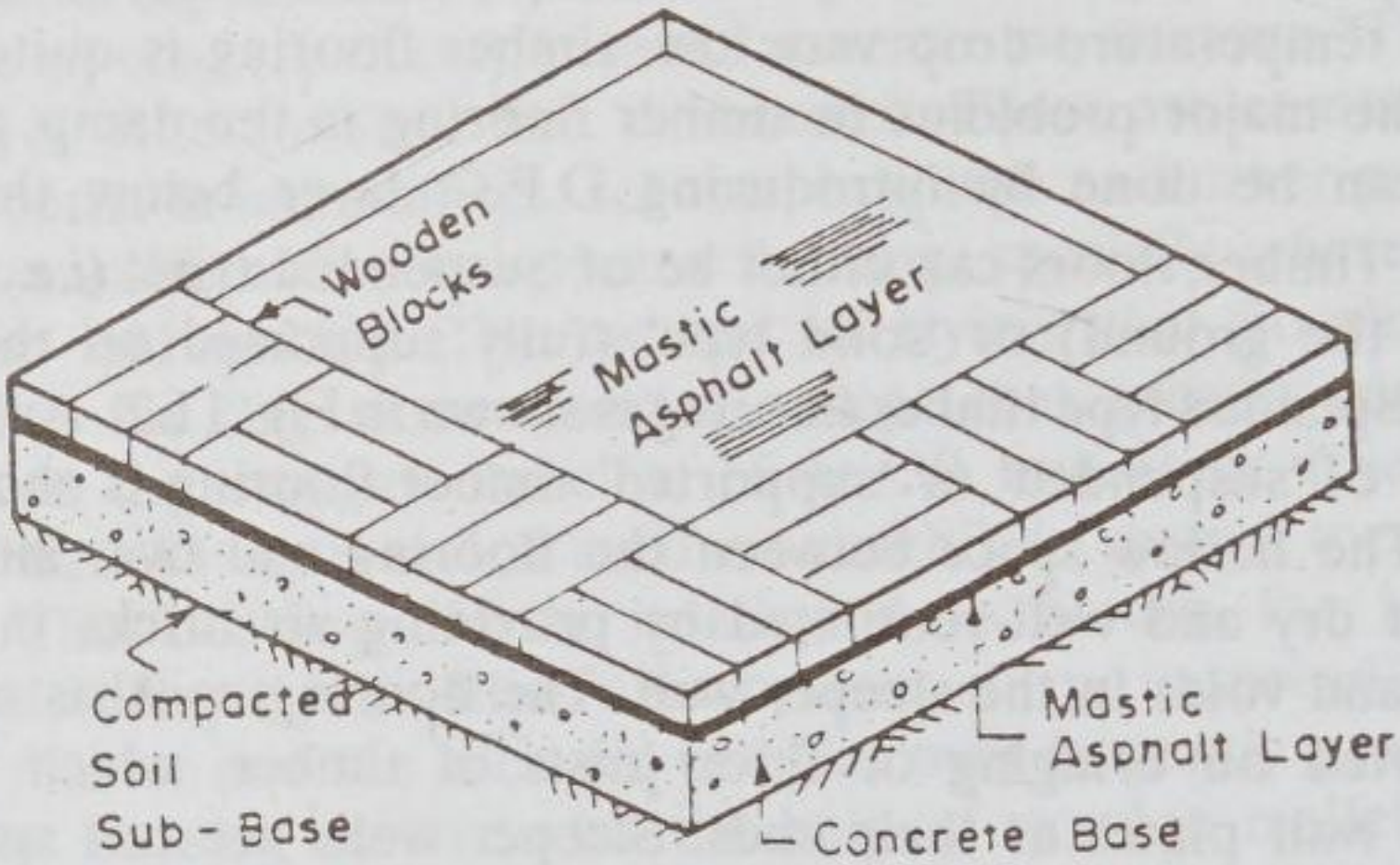


FIG. 11.6. WOODEN BLOCK-FLOORING.

11.14. ASPHALT FLOORING

Asphalt flooring are of many types : (*i*) asphalt mastic flooring (*ii*) asphalt tiles flooring, (*iii*) asphaltic terrazzo, and (*iv*) Acid proof mastic flooring.

1. Asphalt mastic flooring. Asphalt mastic is a mixture of sand (or grit) and asphalt in the ratio of 2 : 1, mixed hot and then laid in continuous sheets. It can also be applied cold, by mixing with mineral oil and asbestos. The thickness of the asphalt mastic may be 2.5 cm for ordinary construction. It is laid on cement concrete base course. The mix is poured on the concrete base, and is spread by means of trowel to get levelled surface. On the top of the surface, a thin layer of sand is spread, which is then rubbed with a trowel. The joints of mastic asphalt laid on successive days are properly lapped.

2. Asphaltic tiles. These are prepared from asphalt, asbestos fibres, inert materials and mineral pigments, by pressing the mix in different sizes (20 cm square to 45 cm square), with thickness varying from 3 to 6 mm. These tiles are either directly cemented to concrete base or are fixed to wooden floors by using an intervening layer of mastic asphalt or asphalt saturated felt. Asphaltic tiles are cheap, resilient, sound proof, non-absorbant and moisture proof.

3. Asphaltic mosaic. This is prepared similar to mastic asphalt, except that marble chips are used in the place of sand/grit. Asphalt may be either in black or other suitable colour, and is laid in hot condition.

4. Acid proof mastic flooring. Acid proof blocks of asphalt are available, which are manufactured from moulding acid proof asphalt and inert crushed rock aggregate under high pressure. The asphalt blocks are first laid on concrete base then acid proof asphalt is uniformly spread over the surface of the blocks. Find sand is spread over the liquid asphalt before it hardens.

11.15. RUBBER FLOORING

It consists of sheets or tiles of rubber, in variety of patterns and colours with thickness varying from 3 to 10 mm. The sheet or tile is manufactured by mixing pure rubber with fillers such as cotton fibre, granulated cork or asbestos fibre. The sheets or tiles are fixed to concrete base or wood by means of appropriate adhesives. rubber floorings are resilient and noise proof. However, they are costly.They are used only in office or public buildings.

11.16. LINOLEUM FLOORING (COVERING)

Strictly speaking, it is covering which is available in rolls, and which is spread directly on concrete or wooden flooring. Linoleum sheet is manufactured by mixing oxidized linseed oil in gum, resins, pigments, wood flour, corkdust and other filler materials. The sheets are either plain or printed, and are available in 2 to 6 mm thickness, and 2 to 4 m wide rolls. Linoleum tiles are also available , which can be fixed (or glued) to concrete base or wood floor, in different patterns. Linoleum sheet is either spread as such, or also may be glued to the base by inserting a layer of saturated felt. Linoleum covering are attractive, resilient, durable and cheap, and can be cleaned very easily. However, it is subjected to rotting when kept wet or moist for some time. It cannot, therefore, be used for bath-rooms, kitchens etc.

11.17. CORK FLOORING

Such type of flooring is perfectly noiseless, and is used in libraries, theatres, art galleries, broadcasting stations etc. Cork, which is the outer bark of cork oak tree, is available in the form of cork carpet and cork tiles. It is fixed to concrete base by inserting a layer of saturated felt. Cork carpet is manufactured by heating granules of cork with linseed oil and compressing it by rolling on canvass. Cork tiles are manufactured from high grade cork bar or shearings compressed in moulds to a thickness of 12 mm and baked subsequently. They are available in various sizes (10 cm × 10 cm to 30 cm × 90 cm), various thicknesses (5 to 15 mm) and various shades.

11.18. GLASS FLOORING

This is a special purpose flooring, used in circumstances where it is desired to transmit light from upper floor to lower floor, and specially to admit light at the basement from the upper floor. Structural

glass is available in the form of tiles or slabs, in thicknesses varying from 12 to 30 mm. These are fixed in closely spaced frames so that glass and the frame can sustain anticipated loads. Glass flooring is very costly, and is not commonly used.

11.19. PLASTIC OR P.V.C. FLOORING

It is made of plastic material, called Poly-Vinyl- Chloride(P.V.C.), fabricated in the form of tiles of different sizes and different colour shades. These tiles are now widely used in all residential as well as non-residential buildings. The tiles are laid on concrete base. Adhesive of specified make is applied on the base as well as on the back of P.V.C. tile with the help of a notched trowel. The tile is laid when the adhesive has set sufficiently (say within 30 minutes of its application); it is gently pressed with the help of a 5 kg weight wooden roller and the oozing out adhesive is wiped off. The floor is washed with warm soap water before use. P.V.C. tile flooring is resilient, smooth, good looking and can be easily cleaned. However, it is costly and slippery, and can be damaged very easily when in contact with burning objects.

PROBLEMS

1. (*a*) Explain, in brief, the essential requirements of a floor.
(*b*) Enumerate various types of flooring materials.

2. Explain the method of laying the following types of flooring :
(*i*) Flag stone flooring. (*ii*) Brick flooring. (*iii*) Marble flooring.

3. Explain the method of constructing cement concrete flooring. What is the use of granolithic finish and how is it made ?

4. Explain the procedure of constructing the following types of flooring:
(*i*) Terrazzo flooring. (*ii*) Mosaic flooring. (*iii*) Tiled flooring.

5. Write short notes on the following types of flooring :
(*i*) Asphaltic flooring. (*ii*) Linoleum flooring. (*iii*) P.V.C. flooring.
(*iv*) Cork flooring. (*v*) Rubber flooring.

6. Explain, with the help of sketches, the method of constructing timber flooring.

7. Explain with reasons what type of floor finishing will be required for
(*i*) Operation theatre. (*ii*) Dancing hall. (*iii*) Library.
(*iv*) Warehouse. (*v*) Factory hall/workshop. (*vi*) Grain store
(*vii*) Testing laboratory. (*viii*) Hostel.

8. Explain in brief the factors that affect the selection of floor a finish.

12

Floors-II : Upper Floors

12.1. INTRODUCTION

An upper floor is basically a principal structural element, and the general structural design of a building will greatly influence the choice of the type of floor. Upper floors are supported either on the walls or on columns ; they have, therefore, the major problems of strength and stability. The structural design of upper floors has to be such as to support the loads set up by the use of the building, in addition to the self weight and the weight of partitions etc. However, the flooring materials are practically the same as used for ground floors (chapter 11).

Depending upon the materials used for construction, and upon the arrangement of beams, girders etc. for supporting the flooring, upper floors may be classified into the following types :

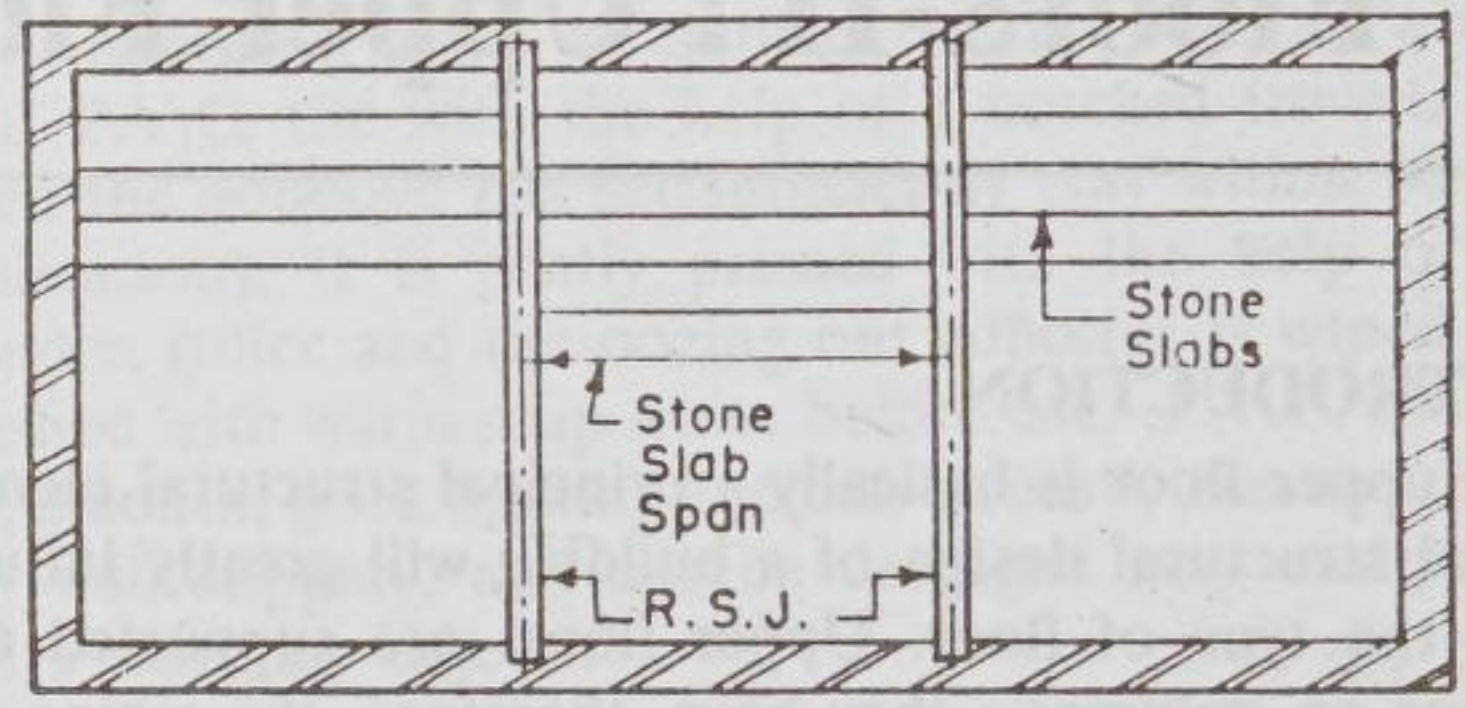
1. Steel joist and stone or precast concrete floors.
2. Jack arch floors.
3. Reinforced cement concrete floors.
4. Ribbed or hollow tiled flooring.
5. Filler joists floors.
6. Precast concrete floors.
7. Timber floors.

12.2. STEEL JOIST AND STONE OR PRECAST CONCRETE SLAB FLOORS.

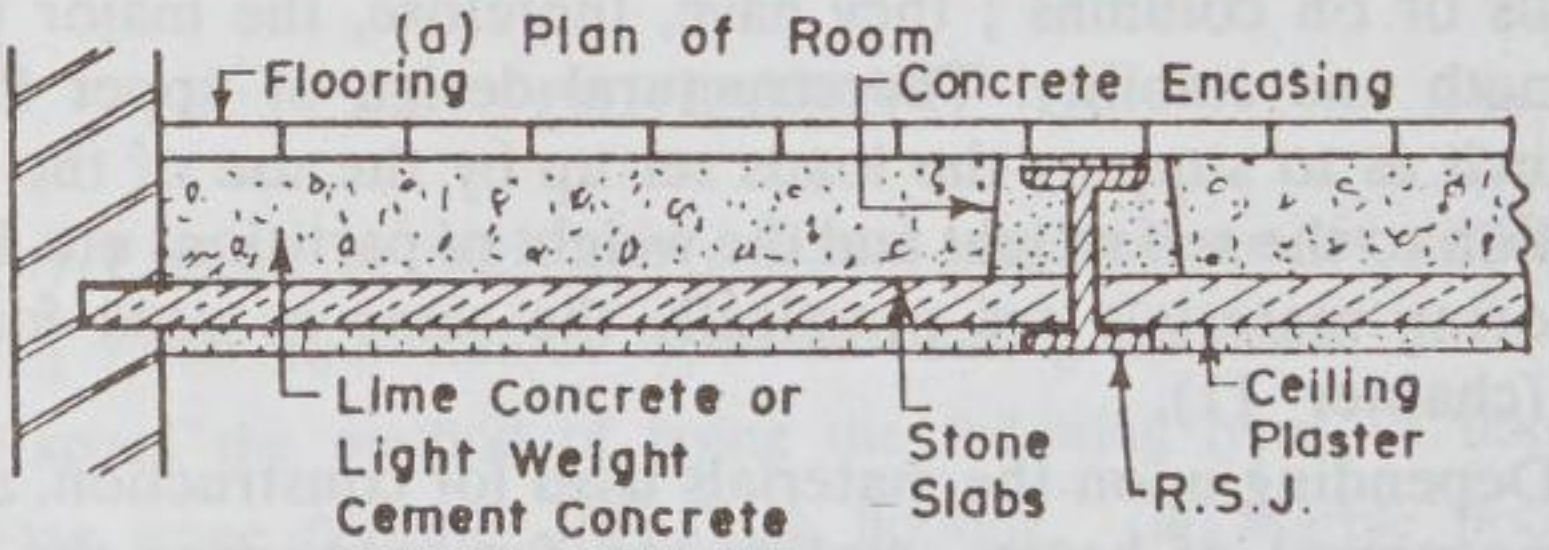
This type of floor is quite common in locations where flag-stones or stone-slabs are readily available in spans of 1 to 3 metres and widths 30 to 60 cm. Where stone slabs are not available, precast concrete slabs can be used . The slabs are placed at the lower flange of rolled steel joists (R.S.J.), specially where plain ceiling is required, though in this case the bearing to the slabs is small. Otherwise, the slabs can be supported on the upper flange of R.S.J. by inserting

wide stone bedding plate, called *suboti* between the flange and the slab (Fig. 12.1 *c*). When the slabs are placed on the lower flange of joists, the space between the top of the slab and top of R.S.J. is filled with lime concrete or light weight cement concrete, after encasing the steel joists completely in cement concrete so that they do not get rusted (Fig. 12.1 *b*). On the top of it, regular flooring is laid.

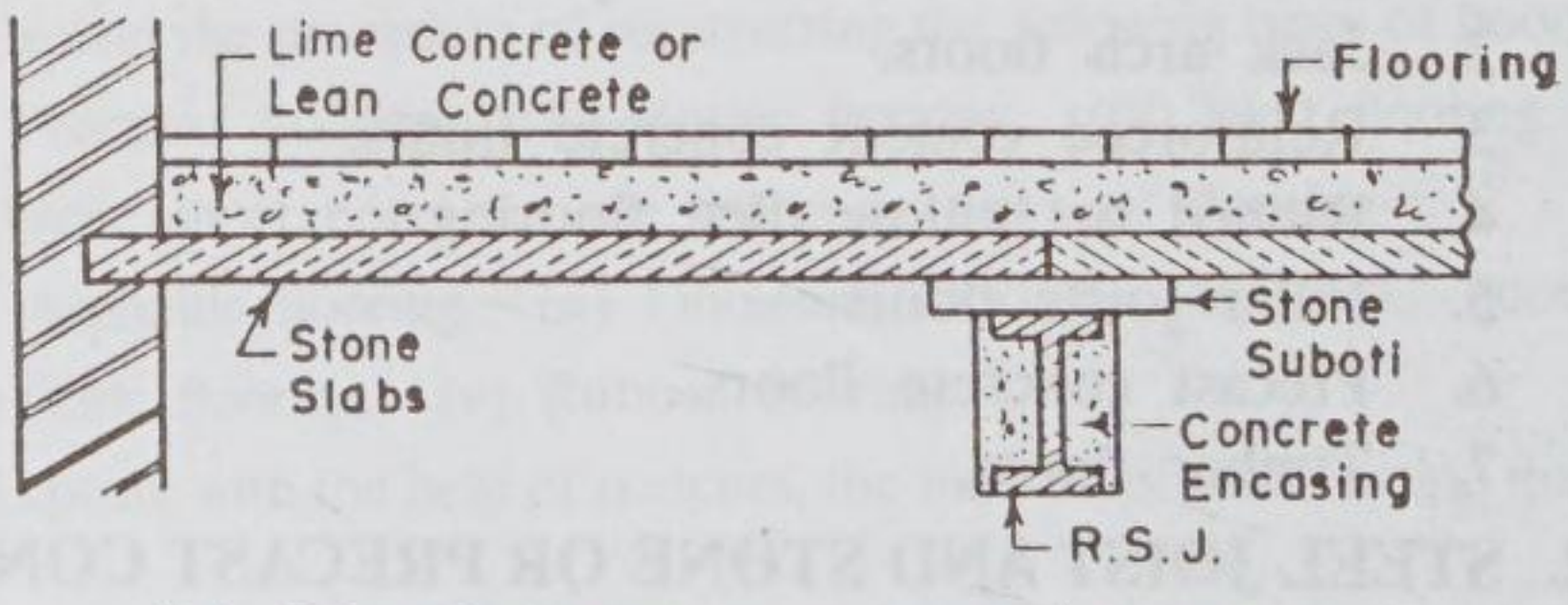
The spacing of the rolled steel joists depend upon the length



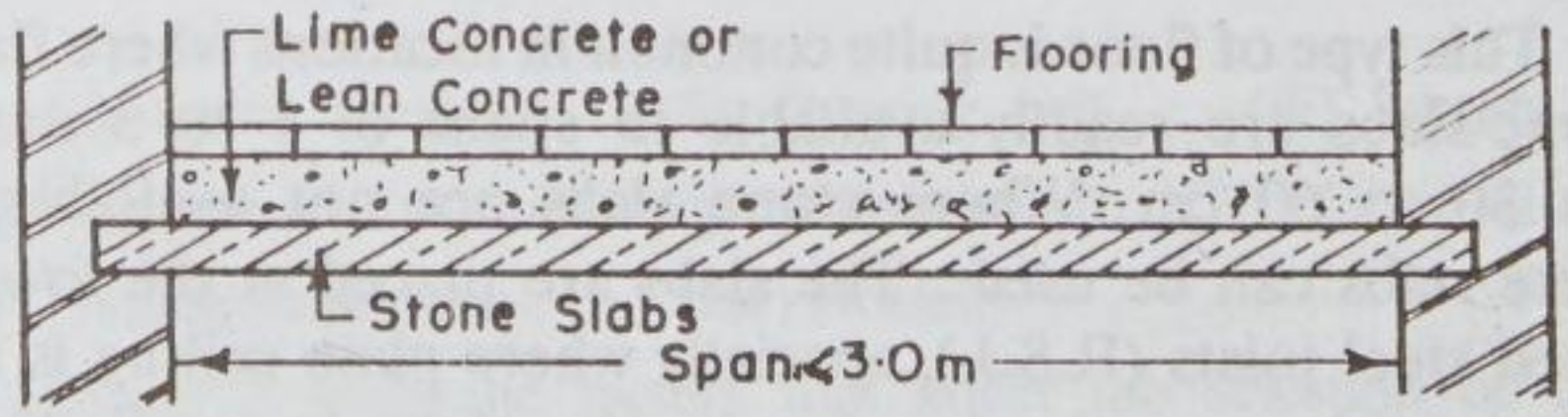
(a) Plan of Room



(b) Stone Slabs on Lower Flange of R.S.J.



(c) Stone Slabs on Upper Flange of R.S.J.



(d) Simple Stone-Slabs Floor

FIG. 12.1. STONE SLAB FLOOR WITH OR WITHOUT JOISTS.

of available stone slabs. The joists have the clear span equal to the width of the room (Fig. 12.1 *a*). The bearing of joists on the wall should at least be equal to depth of the joist, but in no case less than half the width of the wall. It is better if bearing is kept just equal to the width of the wall so that eccentric load of the wall is eliminated. A bed plate is provided below each end of the joist, to suitably distribute the load to the wall.

Sometimes stone slabs are available in lengths of 2.5 to 3.5 m, such as those at Jodhpur. If the width of the room is slightly less than this value, stone slabs can be directly supported on the walls, without using steel joists (Fig. 12.1 *d*). Such a construction is quite cheap.

12.3. JACK ARCH FLOORS

Jack arch is an arch of either brick or concrete, supported on lower flange of mild steel joists (R.S.J.). The joists are spaced 1 to 1.5 m centre to centre, and are supported at their ends either on the walls or on longitudinal girders. The rise of the arch is kept equal to $\frac{1}{12}$th of the span. The minimum depth of concrete at the crown is kept equal to 15 cm. Since the super-imposed load is being borne by arch action, tension is developed on the supporting walls, specially at the end span. Due to this, steel tie rods are provided at the end span, at suitable spacing, usually 1.8 to 2.4 m c/c. The

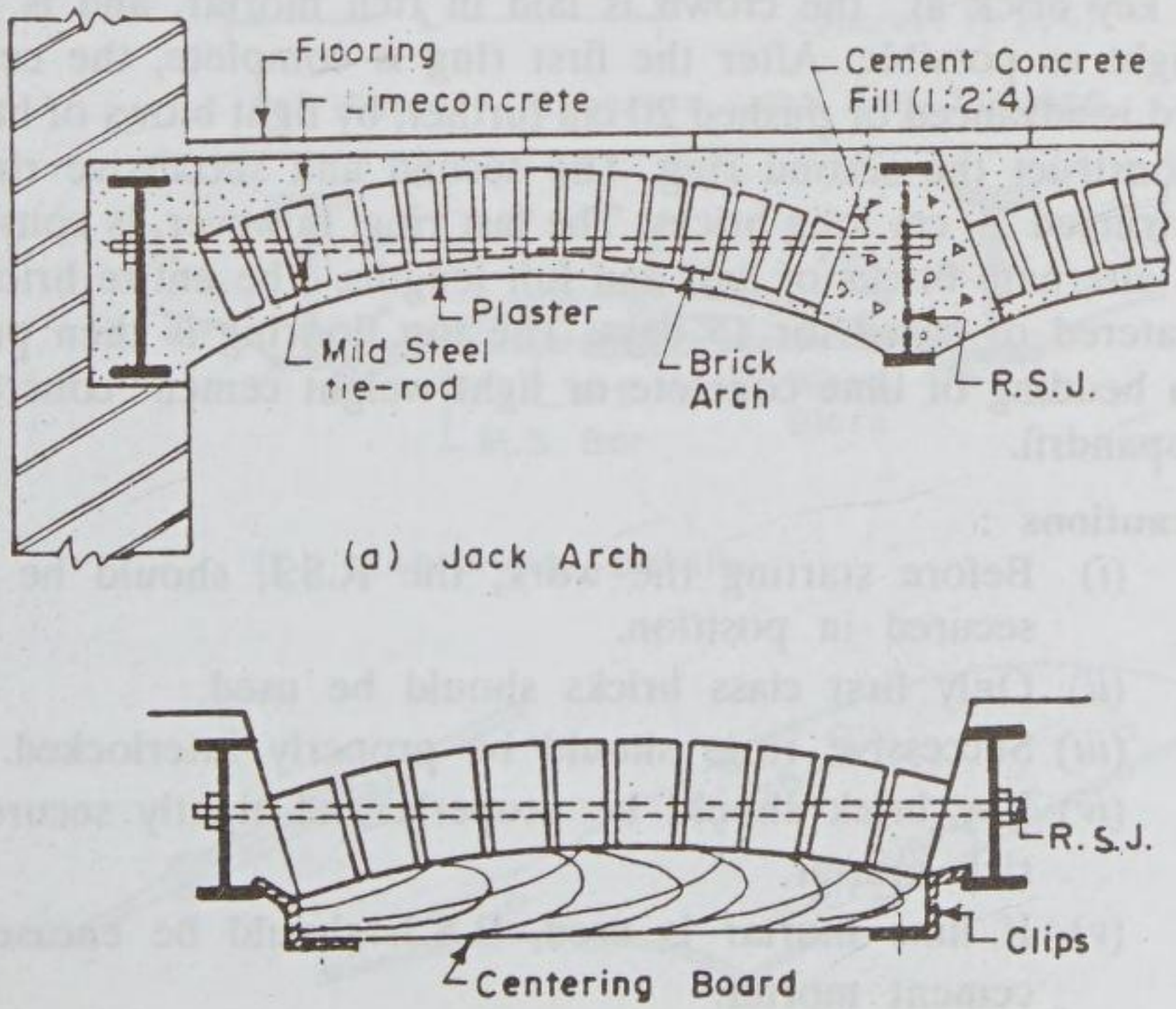


FIG. 12.2. BRICK JACK ARCH FLOORING.

tie rods are 2 to 2.5 cm diameter, and are properly anchored into the wall. The end arch is supported on wall by either providing rolled steel joist into the wall or simply fixing an angle iron or mild steel in the wall. The bottom of the floor is not plane ; this is the only disadvantage of this floor.

Brick Jack Arch Flooring

Fig. 12.2 shows the details of brick jack arch flooring. The construction on jack requires centering of 30 to 40 mm thick segmental piece of timber, with chord length equal to the span of the arch and conforming to the soffit. Then centering board is cut slightly at the ends and is made to rest on the lower flange of R.S.J., with the curved surface upwards. Alternatively, a bend iron strap is attached to its ends to form a hook through which the centering board is suspended from R.J.S., as shown in Fig. 12.2 (*b*). After the centering is ready, bricks are laid on edge from both the joists. The end bricks are cut suitably to fit firmly with the joists. Only well-burnt bricks are used for the construction, and they are saturated with water, before use. Joists are encased in cement mortar, so as to prevent their rusting from lime mortar. The bricks are laid in such a way that necessary bond is developed between different rings or layers of bricks. In the first ring, the bricks are laid in lengths of 20 cm and 10 cm alternatively, to secure good bond between this ring and the next ring along the length of arch (perpendicular to the span). The key brick at the crown is laid in rich mortar, and is pushed as tight as possible. After the first ring is complete, the centering board is advanced or pushed 20 cm further, by light blows of hammer, to construct the second ring, The second and successive rings are constructed 20 cm long bricks. The last ring, however, is constructed with alternate bricks of half and full lengths. The entire brick work is watered or cured for 15 days. The top flooring is then provided on a bedding of lime concrete or light weight cement concrete put on spandril.

Precautions :

(*i*) Before starting the work, the R.S.J. should be properly secured in position.

(*ii*) Only first class bricks should be used.

(*iii*) Successive rings should be properly interlocked.

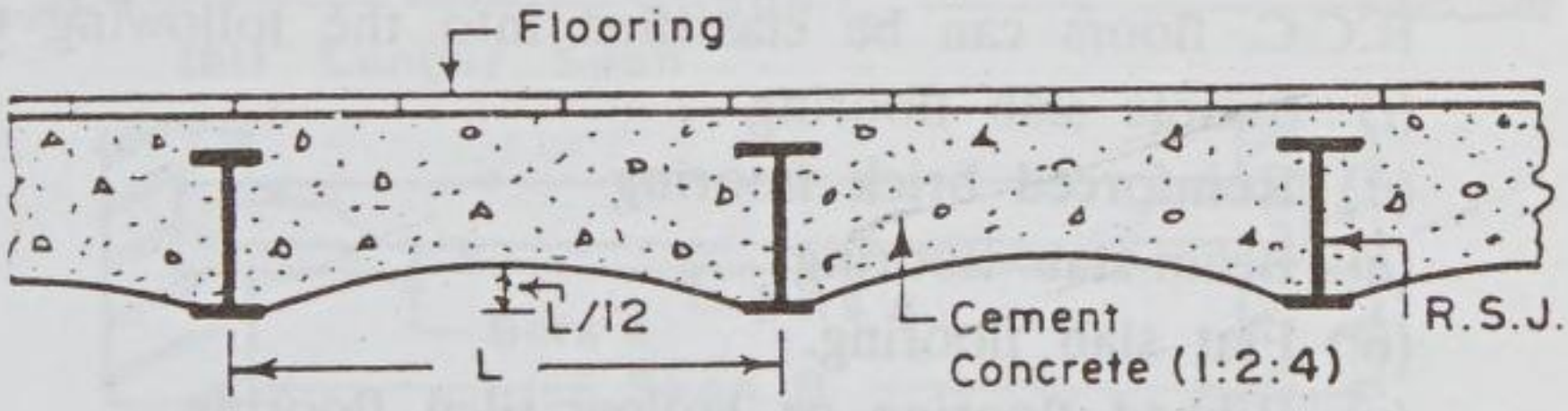
(*iv*) Key brick should be properly and tightly secured in rich mortar.

(*v*) If lime mortar is used, R.S.J. should be encased in cement mortar.

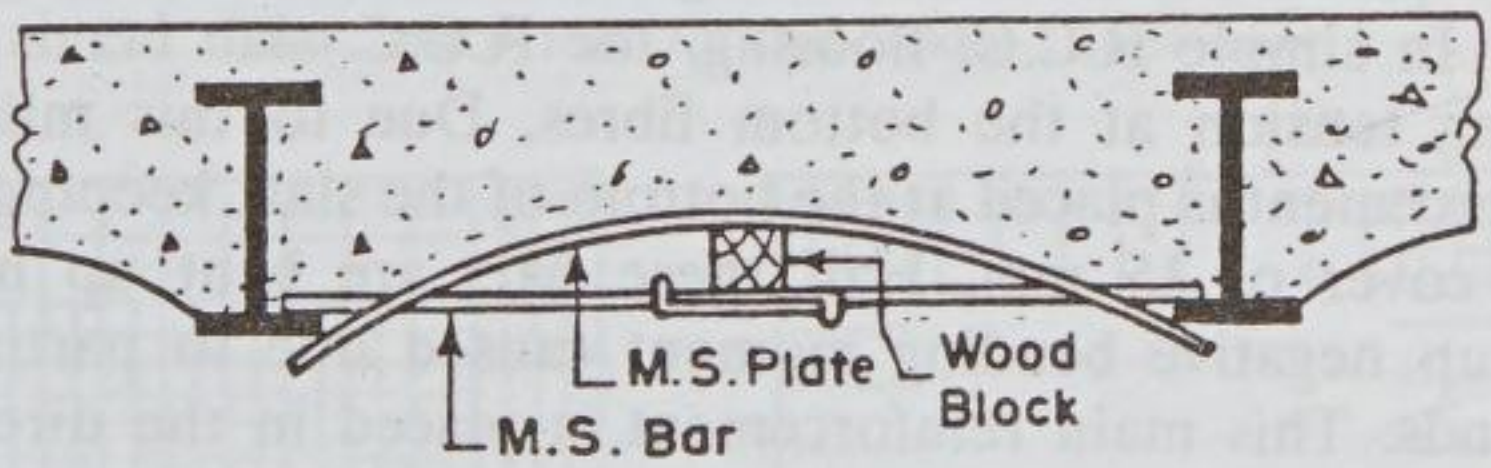
(*vi*) Top concrete and flooring should not be laid unless the brickwork is properly cured.

Cement Concrete Jack Arch Flooring

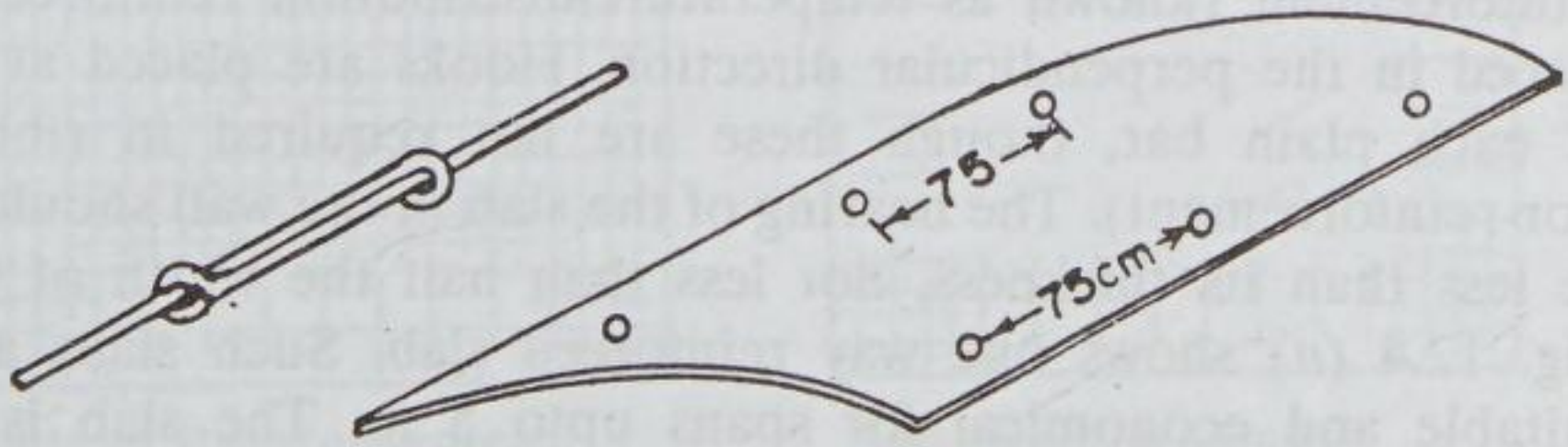
Fig. 12.3 (*a*) shows a cement concrete jack arch flooring in which the arches are made of 1 : 2 : 4 cement concrete, supported on the lower flanges of M.S. joints. The construction of concrete jack arches is relatively simple. The centering consists of a 3 mm thick mild steel plate, bent to the shape of arch soffit, and having pair of holes at ends, spaced at 75 cm c/c. The centering plate is supported on the lower flange of joists through a pair of 12 mm dia. rods, each having an eye hook at its end (Fig. 12.3 *b*). Each rod passes through the end eye of the other (Fig. 12.3 *c*), and their total length is adjusted to the span of the arch. The ends of the rods pass through symmetrical holes of the centering plate (Fig. 12.2 *d*) and finally rest on the lower flange of R.S.J., thus providing the support to the M.S. plate, as shown in Fig. 12.3 (*a*). In order to check the deflection of the centering plate, a wooden packing block is tightly inserted between the M.S. plate and the rods. When the centering is ready, cement concrete of 1 : 2 : 4 mix is laid on the top of the M.S. plate, to the required depth



(a) Cement Concrete Jack arch Flooring



(b) Centering Details



(c) Eye Link (d) 3mm thick m.s. Centering Plate

FIG. 12.3. CEMENT CONCRETE JACK ARCH FLOORING.

and is properly compacted either manually or with the help of a vibrator. The flooring is then completed with the desired type of flooring material. The entire work is then well watered for 10 days, for efficient curing. After that, the centering is removed by first removing the wooden packing and then hammering the eyes of the rods toward each other. The under side of the arches can be plastered to give good appearance.

12.4. REINFORCED CEMENT CONCRETE FLOORS

Floors of modern buildings are invariably made of reinforced cement concrete (R.C.C.), because of the inherent advantages of this type of construction. Concrete, though strong in compression, is weak in tension . However, it is suitably reinforced with the help of mild steel bars which take the entire bending tension. Due to this, the overall thickness of R.C.C. floors is comparatively small, thereby reducing the self weight of floor itself. R.C.C. floors are also comparatively fire proof and damp proof. The method of construction is also easy except that centering is required. These floors can also be used on large spans, and therefore, more suitable for big size rooms, halls etc.

R.C.C. floors can be classified into the following types :

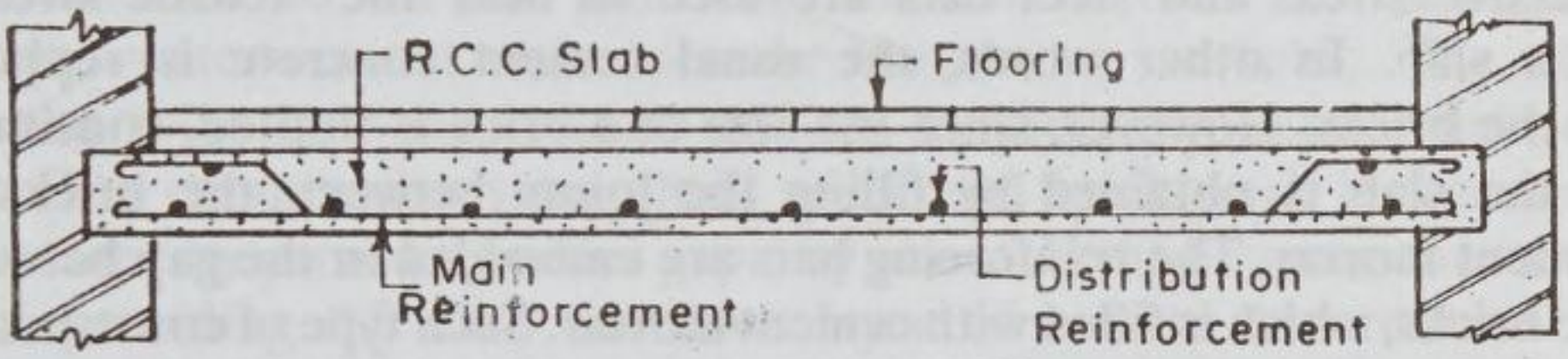
(*i*) Simple slab flooring
(*ii*) Reinforced brick flooring
(*iii*) Beam-slab flooring
(*iv*) Flat slab flooring.
(*v*) Ribbed flooring or hollow tiled flooring.

1. Simple R.C.C. slab Flooring

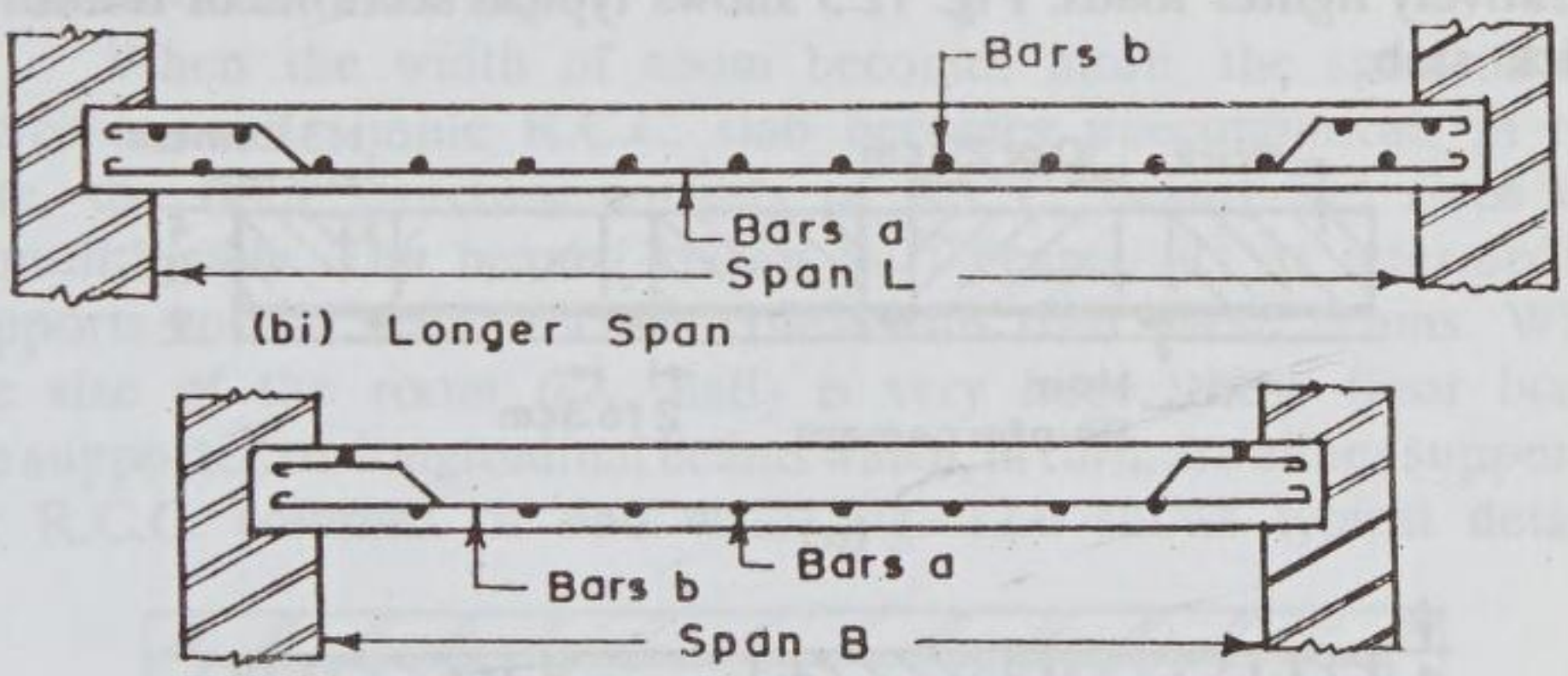
In simple R.C.C. flooring, the R.C.C. slab bends downwards, causing tension at the bottom fibres. Due to this mild steel bars reinforcement is placed at the bottom of the slab, keeping a minimum clear cover of 15 mm. Half these bars are bent up near ends to take up negative bending moment caused due to partial fixidity at the ends. This main reinforcement is placed in the direction of the span of the slab, which is equal to the width of the room, specially when the length of the room is more than 1.5 times the width of the room. Such a slab is known as *one way reinforced slab.* Nominal reinforcement (known as temperature/distribution reinforcement) is placed in the perpendicular direction. Hooks are placed at the end of each plain bar, though these are not required in ribbed bars (tor-reinforcement). The bearing of the slab in the wall should neither be less than its thickness, nor less than half the width of the wall. Fig. 12.4 (*a*) shows one way reinforced slab. Such slabs are quite suitable and economical for spans upto 5 m. The slab is cast on timber or steel shuttering. After erecting the centering, properly bent reinforcement is placed in position. Distance pieces of stone or concrete

are placed between the reinforcement and the shuttering plate so that proper cover is maintained. Cement concrete of appropriate mix (usually 1 : 2 : 4) is then poured and well-compacted. The slab is then properly cured. Shuttering is removed only when the concrete has fully set.

When the length of the room is less than 1.5 times the width of the room, the slab spans/bends in both the directions. It is essential



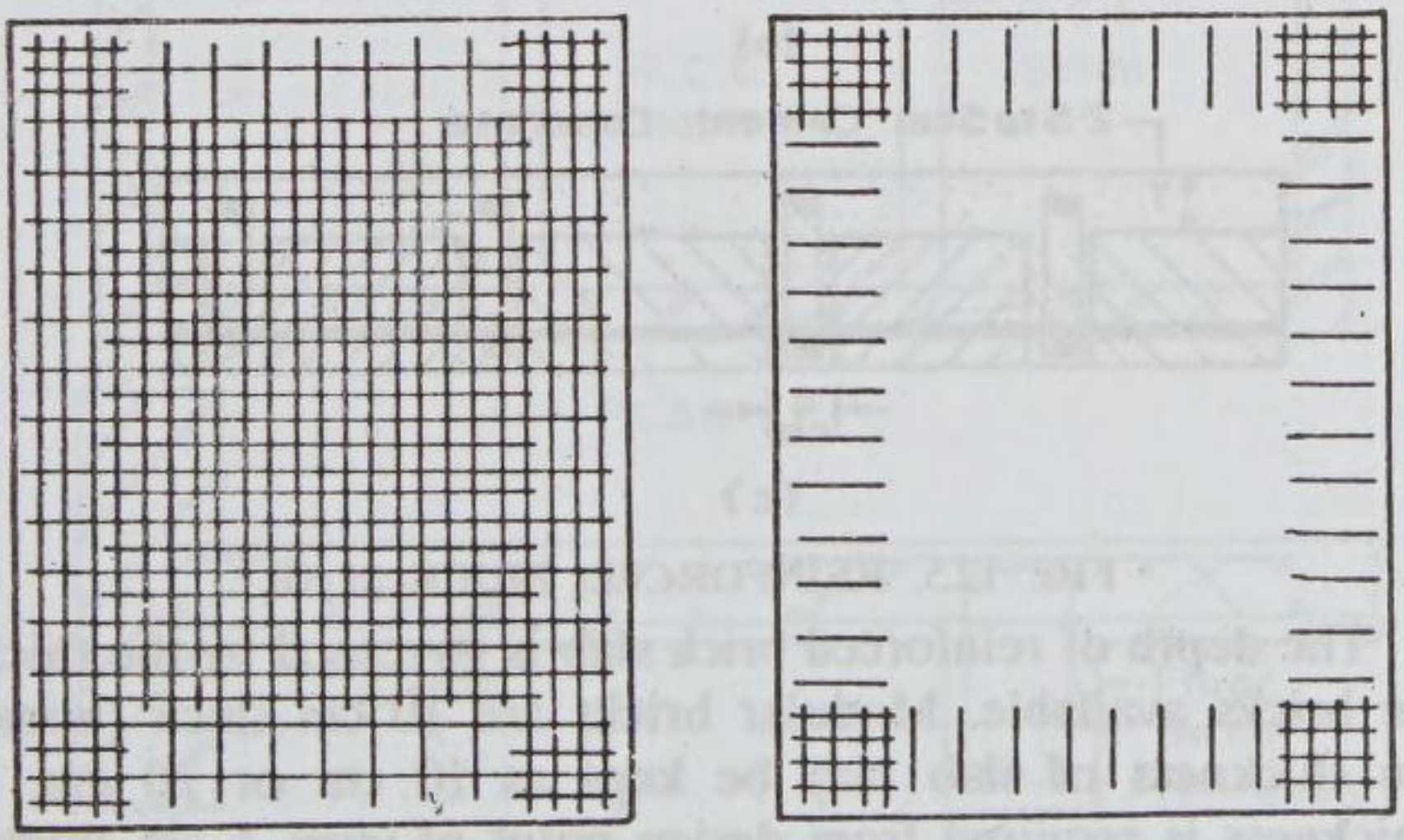
(a) One Way Reinforcement



(bi) Longer Span

(bii) Shorter Span

(b) Two Way Reinforced Slab (Sections)



Bottom Reinforcement Top Reinforcement

(c) Two Way Reinforced Slab (Plans)

FIG. 12.4. REINFORCED CONCRETE SLABS.

to provide reinforcement in both the directions. Such a slab is known as a *two-way reinforced slab,* such as the one shown in Fig. 12.4(*b*). At the corner, suitable mesh reinforcement is provided at the top and bottom, to prevent their lifting. The plan of the reinforcement of a two-way slab, at its top and bottom is shown in Fig. 12.4 (*c*).

2. Reinforced Brick Flooring

Reinforced brick work is a typical type of construction in which the compressive strength of bricks is utilised to bear the compressive stress and steel bars are used to bear the tensile stresses in a slab. In other words, the usual cement concrete is replaced by the bricks. However, since the size of a brick is limited, continuity in the slab is obtained by filling the joints between the bricks by cement mortar. The reinforcing bars are embedded in the gap between the bricks, which is filled with cement mortar. Such type of construction is quite suitable and cheap for small span floor slabs carrying comparatively lighter loads. Fig. 12.5 shows typical sections of reinforced brick slab.

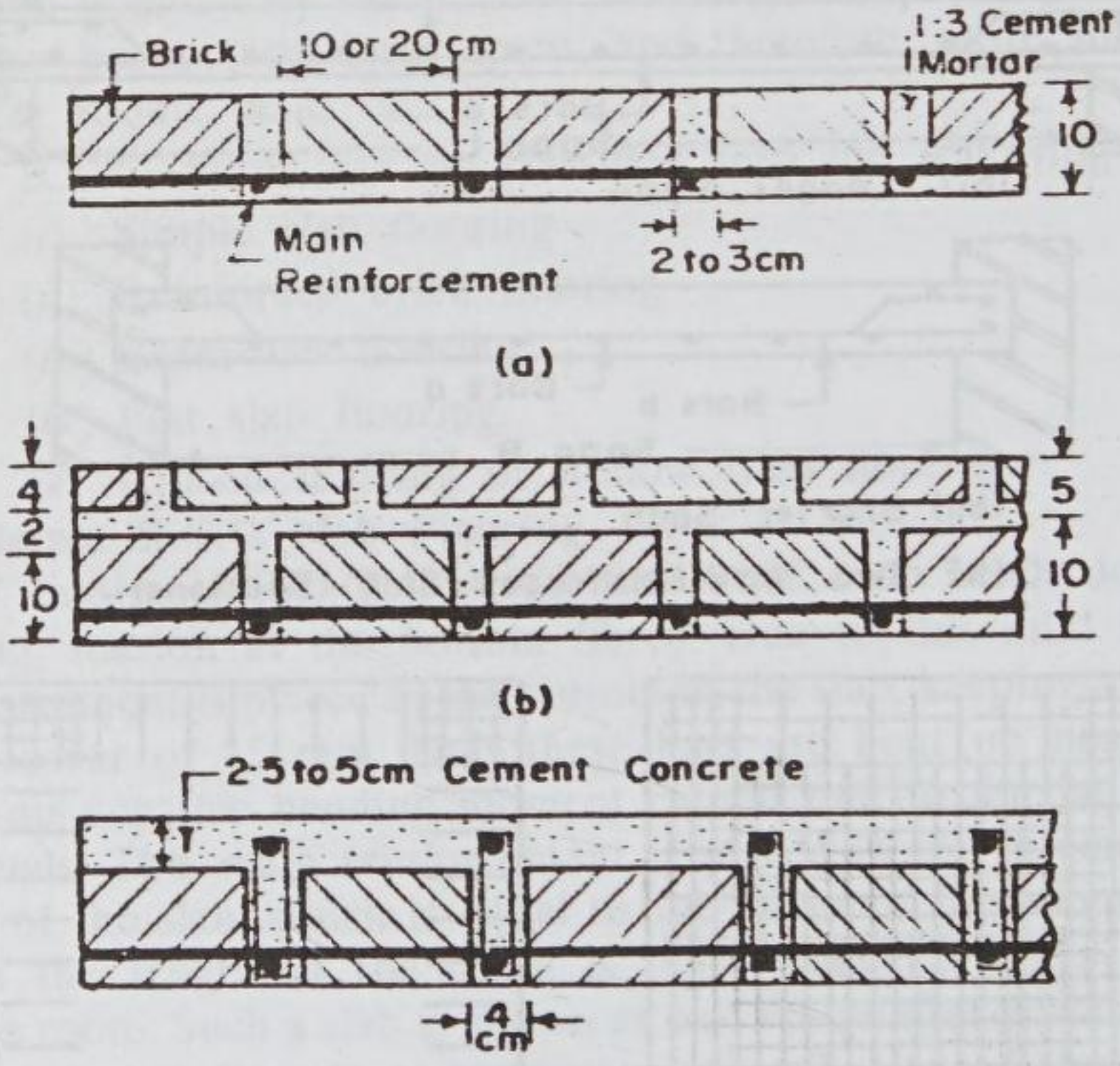


FIG. 12.5. REINFORCED BRICK SLAB.

The depth of reinforced brick slab is governed by the thickness of the bricks available. Modular bricks are 10 cm thick (nominal). Hence thickness of slab may be kept as 10 cm or 20 cm. If 15 cm thickness is required from design point of view, 5 cm thick tiles are used on the 10 cm thick bricks to make a total thickness of 15 cm (Fig. 12.5 *b*). The joint between the two layers of tile and

brick is filled with cement mortar. Before use, the bricks should be thoroughly soaked in water. The reinforcing bars put in the joints should not come in contact with bricks.

When two layers of bricks are used, vertical joints in the bricks should be broken (staggered) so that slab does not shear along the joint. The bricks near the edge should rest half on the bearing wall so that vertical joint above the edge of the wall is avoided. First class bricks should be used for such a work. Cement mortar used to fill the joints etc. should be of 1 : 3 ratio, with proper water-cement ratio to make the mortar workable. The width of the joint between adjacent bricks is generally kept equal to 2 cm. The compressive strength of reinforced brick work is sometimes increased by providing wider gap (say about 4 cm) between the bricks, and providing 2.5 to 5 cm thick layer of cement concrete on the top of the bricks, as shown in Fig. 12.5 (*c*).

3. Beam-Slab Flooring

When the width of room becomes more, the span of slab increases, and simple R.C.C. slab becomes uneconomical. In that case, the floor structure consists of R.C.C. beams and slabs cast monolithically. The beams, known as T-beams, act as intermediate supports to the slab which is continuous over these beams. When the size of the room (*i.e.*. hall) is very large, these floor beams are supported on longitudinal beams which, in turn, are either supported on R.C.C. columns or end walls Fig. 12.6 shows typical details.

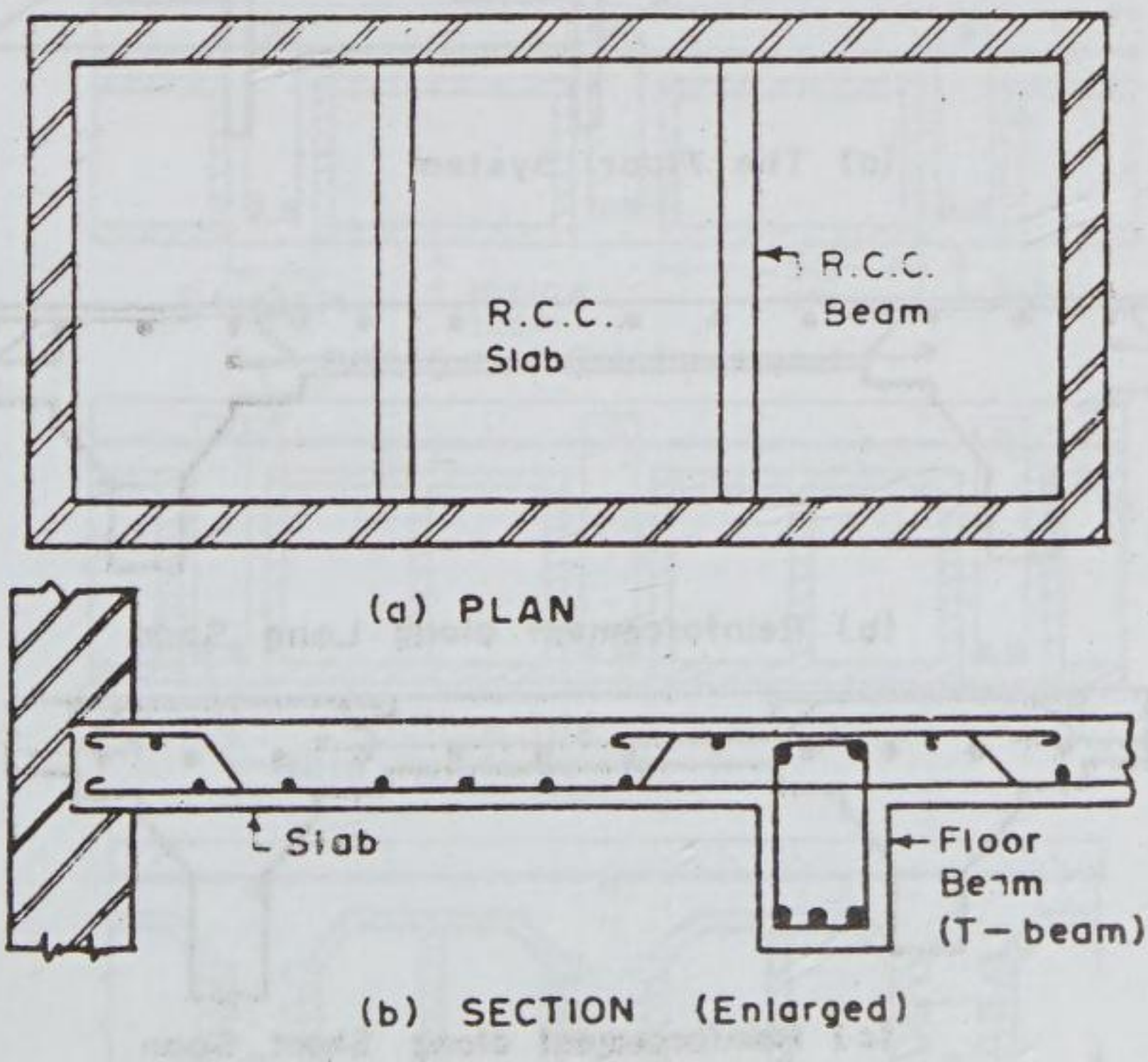
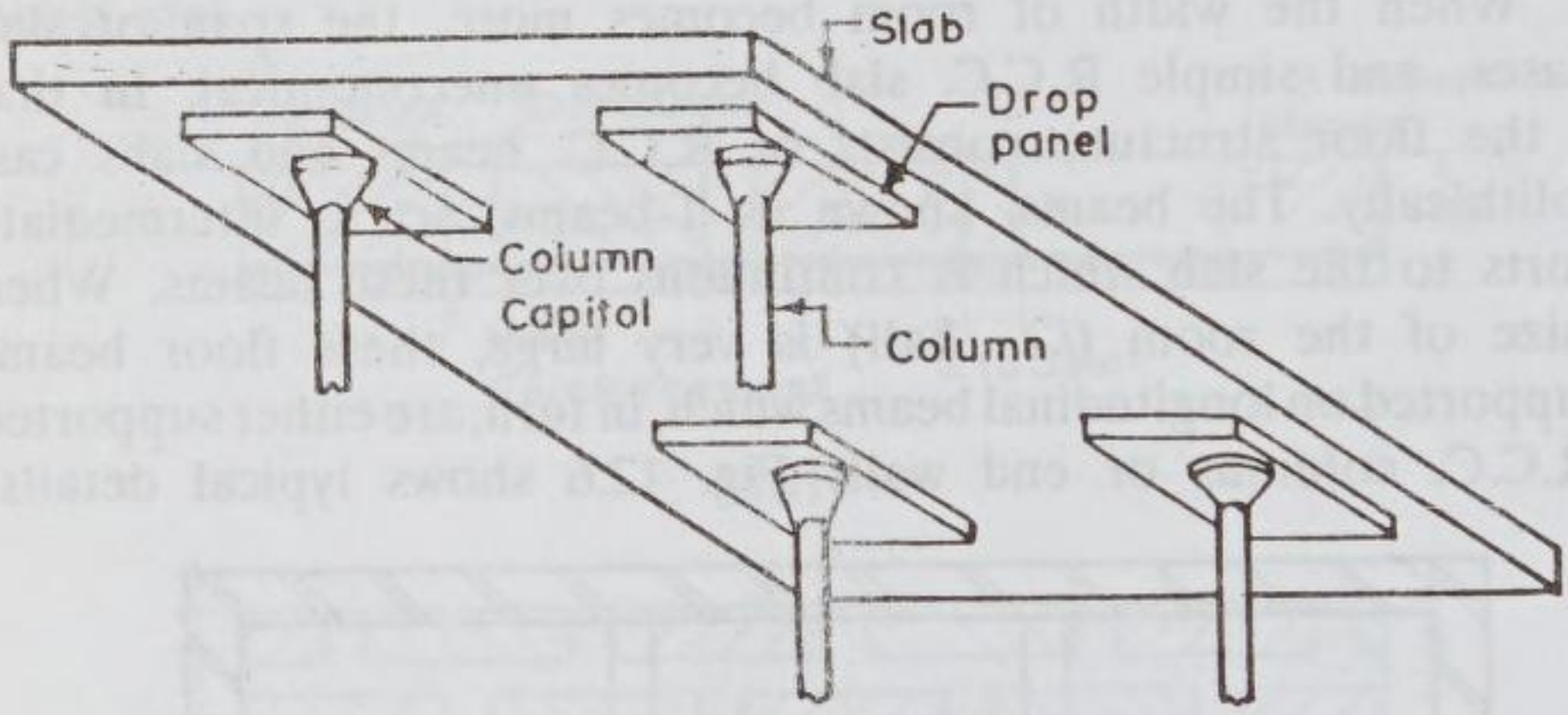


FIG. 12.6. BEAM-SLAB FLOORING.

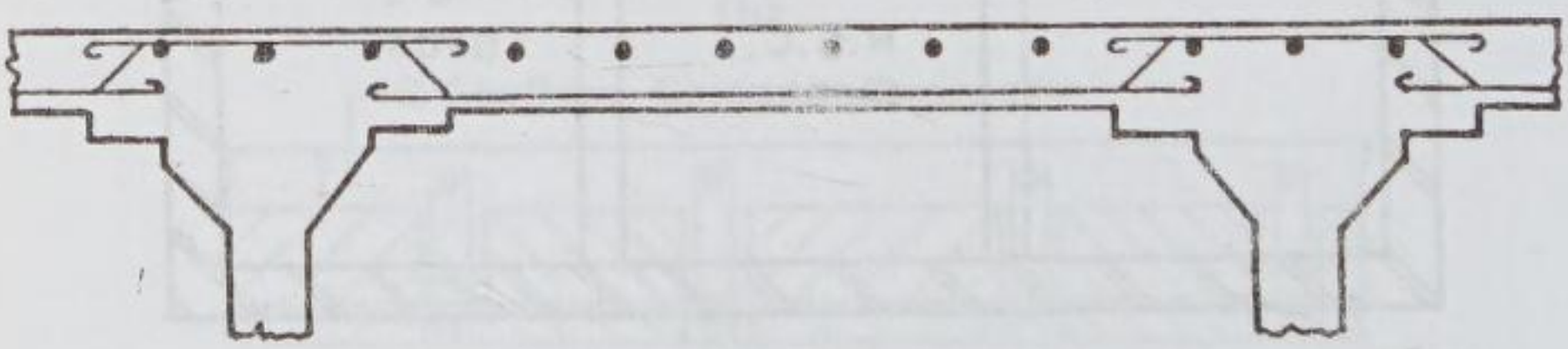
4. Flat Slab Flooring

A flat slab is a typical type of construction in which a reinforced slab is built monolithically with the supporting columns and is reinforced in two or more directions, without any provision of beams. The flat slab thus transfers the load directly to the supporting columns suitably spaced below the slab (Fig. 12.7). Because of exclusion of beam system in this type of construction, a plain ceiling is obtained, thus giving attractive appearance from architectural point of view. The plain ceiling diffuses the light better and is considered less vulnerable in case of fire than the usual beam slab construction. Concrete is more logically used in this type of construction, and hence in case of large spans and heavy load, the total cost is considerably less.

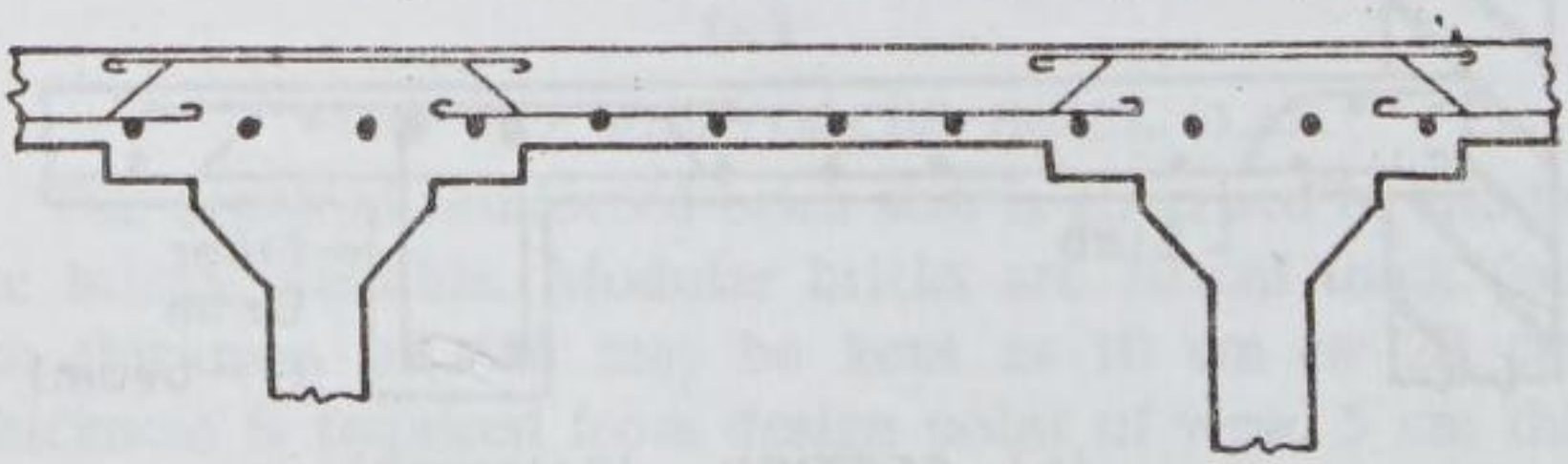
The slab in a flat slab construction may be either with drop or without drop. Drop is that part of the slab around the column



(a) The Floor System



(b) Reinforcement along Long Span



(c) Reinforcement along Short Span

FIG. 12.7. FLAT SLAB CONSTRUCTION.

which is of greater thickness than the rest of the slab. Reinforcement in the slab can be arranged either in two-way system or in four-way system. Two-way system of reinforcement is commonly adopted for slab subjected to ordinary loading conditions. Fig. 12.7 (*b*) and (*c*) shows details of reinforcement in the slab along two directions, in the two-ways system.

12.5. RIBBED OR HOLLOW TILED FLOORING

Concrete is incapable of resisting tension which is caused in the lower part of the thickness of the slab. This lower part does not partake in load bearing, and hence part of it can be replaced by hollow tiles so that weight of the slab is reduced. This results in a *ribbed floor system*, as shown in Fig. 12.8. Unlike T-beam construction, the ribs of hollow tile construction are closely spaced. The clear spacing of ribs depends upon the size of hollow blocks available, but it should normally not exceed 50 cm. The width of ribs may vary between 6 to 10 cm. The span of ribs may be as much as 7 m. However, when the span exceeds 3 m, lateral ribs of the same width as the main longitudinal ones are provided at intervals between 1 to 3 metres. In that case, longitudinal ribs are designed as continuous beams. Main reinforcement is provided at the bottom of the rib. To resist the support moment (negative) an additional bar is placed at the top of rib section. A minimum cover of 2.5 cm is provided. The depth of rib is calculated on the

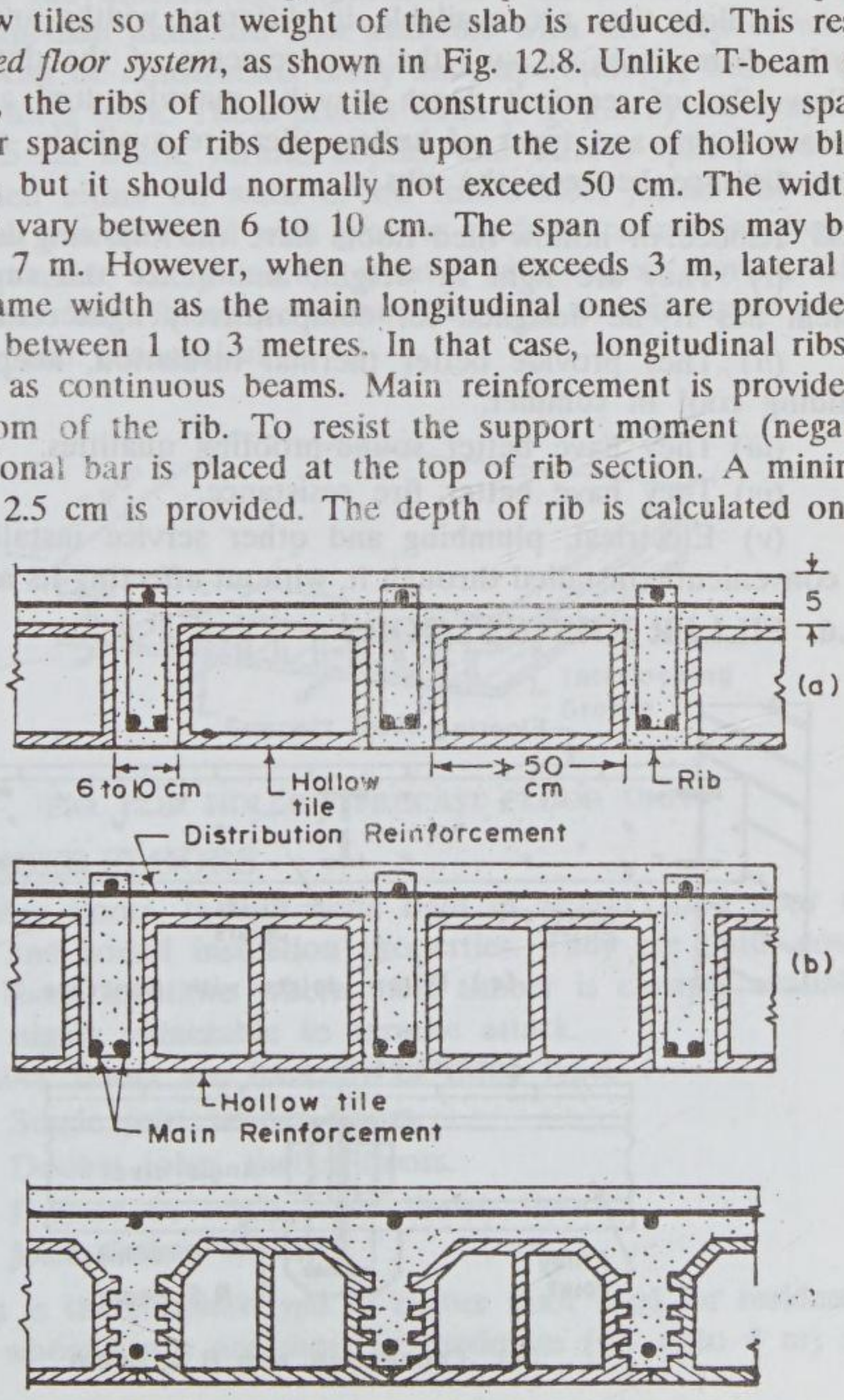


FIG. 12.8. HOLLOW TILE AND CLOSE-RIBBED FLOORS.

basis of bending moment as well as the cost ratio of steel and concrete. Depth of rib is usually kept as at least $L/20$ with free support and at least $L/25$ with fixed support, where L is the span of the ribs.

Due to small span, the slab is normally not analysed. Slab thickness of 4 to 5 cm is generally provided. To check its cracking and to distribute the load properly, shrinkage and temperature reinforcement is provided in the slab, in both the directions. Sometimes, a welded fabric is arranged approximately along the middle of the thick: .s of the slab.

Hollow tiles are available in different widths and different depths. Sometimes, to suit the requirements of the depth of rib, hollow tiles of required depth may be manufactured at the site. Various forms and types of hollow tiles are available, so suit the clear distance between the ribs.

Ribbed or hollow-tiled floors have the following advantages :

(*i*) They are light in weight, and hence the supporting system has to be designed for comparatively lighter loads.

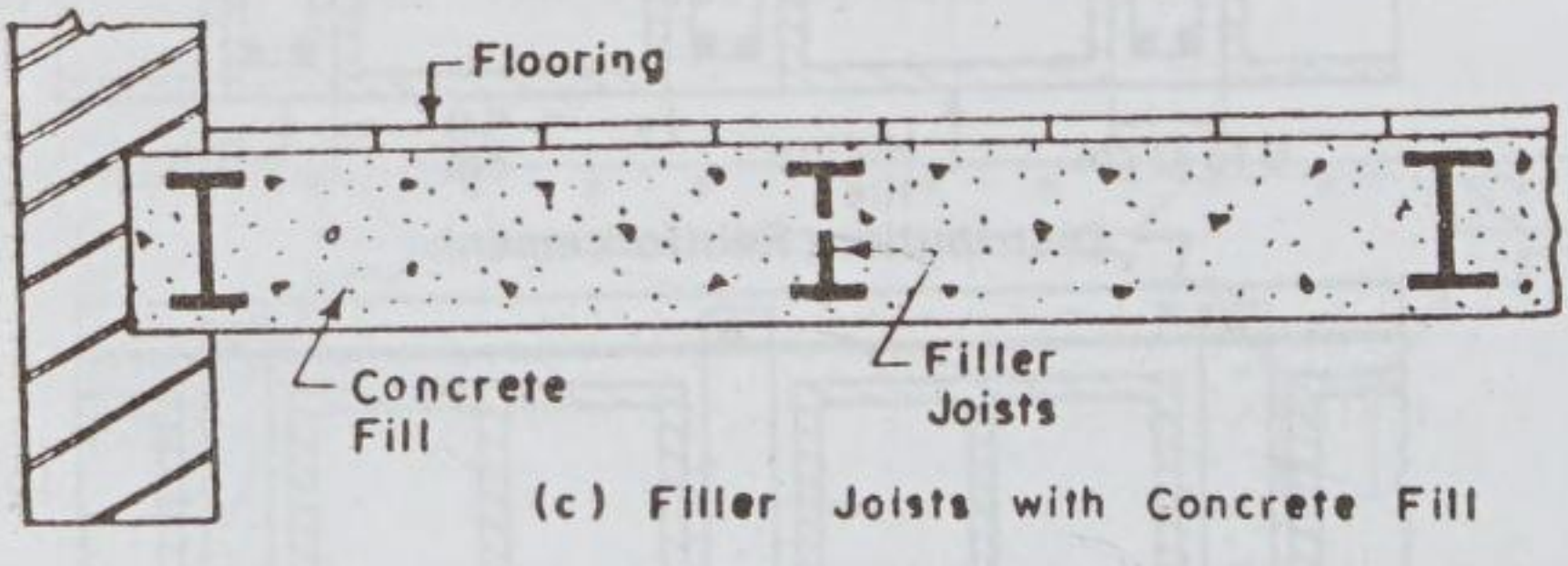
(*ii*) They provide better thermal insulation, keeping the building cool in summer.

(*iii*) They have better sound-proofing qualities.

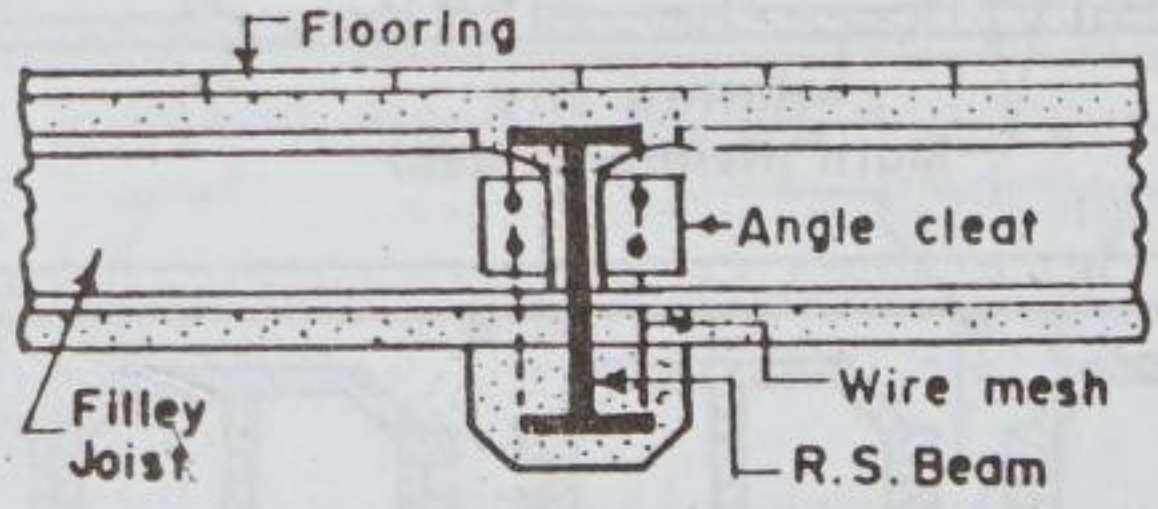
(*iv*) They have better fire resistance.

(*v*) Electrical, plumbing and other service installations can be conveniently installed through it, without affecting its appearance.

12.6. FILLER JOISTS FLOORS



(c) Filler Joists with Concrete Fill



(b) Connection with R.S. Beam

FIG. 12.9. FILLER JOISTS FLOOR.

This is a typical type of composite construction in which R.S.J. of small sections are placed in concrete, as shown in Fig. 12.9. The spacing of the joists may vary between 40 to 90 cm. The *filter joists* may either rest on walls (if the span is less) or on longitudinal steel beams. The joists act as reinforcement, and no separate reinforcement is provided in the concrete filled in between the joists. Concrete should completely surround the filler joists and steel beams, with a minimum cover of 2.5 cm over filler joists.

12.7. PRE-CAST CONCRETE FLOORS

With the modern developments in construction technology, precast beam-slab units are now available with the help of which the floors can be constructed easily and expeditiously, without the aid of any form work. These precast units (Fig. 12.10) are available in about 25 cm width, various depths, and various spans, and can be supported either on walls or on rolled steel joists. The sides of each unit are provided with grooves to form connecting joggles for adjacent units. The joints are grouted with cement mortar, using concrete guns. Such floors are economical, light weight, sound proof, fire proof, and economical.

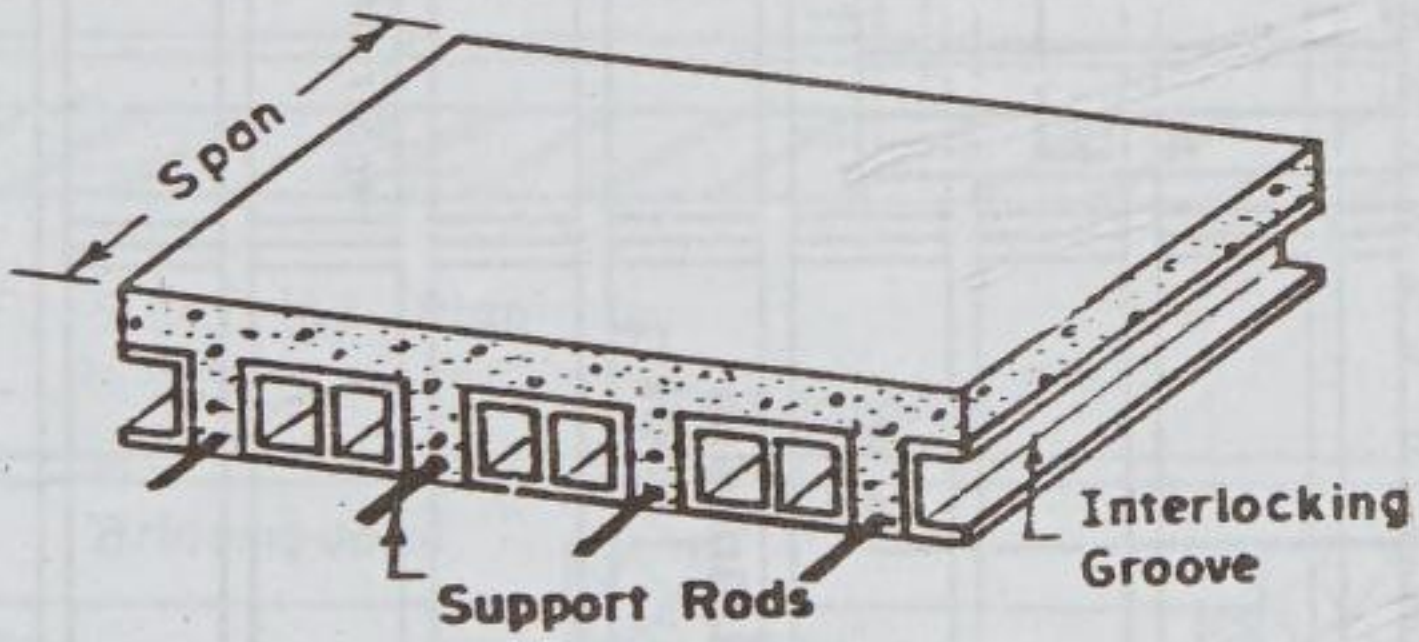


FIG. 12.10. HOLLOW PRECAST FLOOR UNITS.

12.8. TIMBER FLOORS

Timber floors, though quite light in weight, have poor fire resistance and sound insulation properties. They are quite costly, except at those locations where local timber is cheaply available. It is also highly vulnerable to termite attack.

Timber floors are basically of three types :

1. Single joist timber floors.
2. Double joists timber floors.
3. Framed or triple joists timber floors.

1. Single joist timber floors.

This is the simplest type of timber floor used for residential buildings, where spans are short or moderate (say upto 4 m) and

loads are comparatively lighter. The floor consists of *wooden joists* (also called *bridging joists*) spaced 30 to 40 cm apart and supported on end walls, over which timber planking or boarding is fixed. The width of joists are kept 5 to 8 cm wide. The depth of the joists is determined from the thumb rule :

$$\text{Depth (cm)} = (4 \times \text{span in metres}) + 5 \text{ cm.}$$

The joists are supported on wall plates 10 × 7 cm to12×7 cm in size, at the end walls. A space of about 5 cm is kept at the ends for air circulation. When the span exceeds 2.5 m, it becomes essential to strengthen the timber joists by providing *herring bone strutting* at the mid-span, by means of inclined pieces of timber of size 5×3 cm to 5×5 cm, as shown in Fig. 12.11 (*c*). End wedges are provided between the wall and joist.

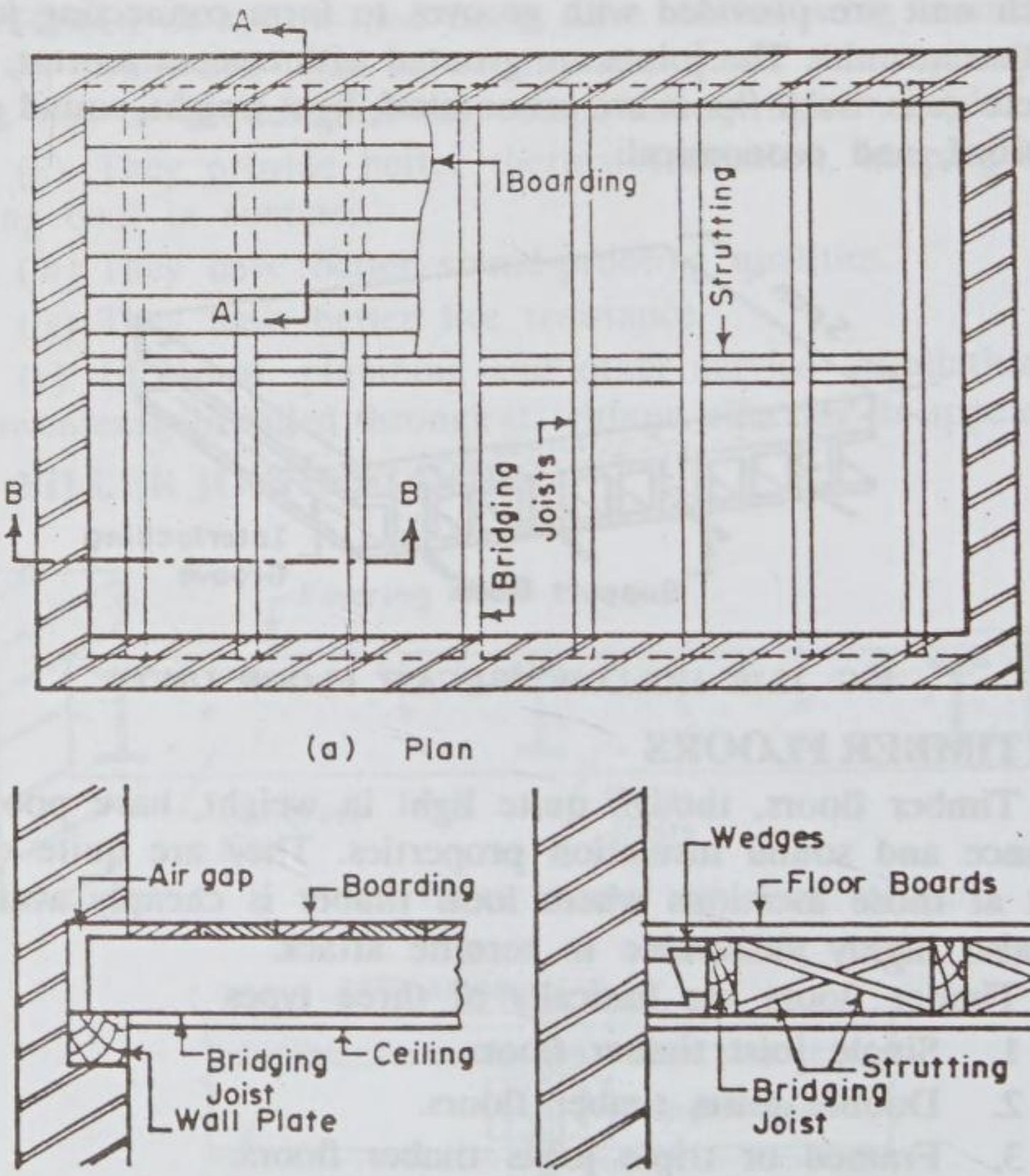


FIG. 12.11. SINGLE JOISTS TIMBER FLOORING.

The end of the joists are nailed, cogged or notched to the wall plates. If the joists of adjacent room run in the same direction, they may be overlapped and nailed to each other. Planking consists of wooden boards of 4 cm thick and 10 to 15 cm width, which are fixed to the bridging joists.

2. Double joists timber flooring

This type of flooring is stronger, and is used for spans between 3.5 to 7.5 metres. The bridging joists are supported on intermediate

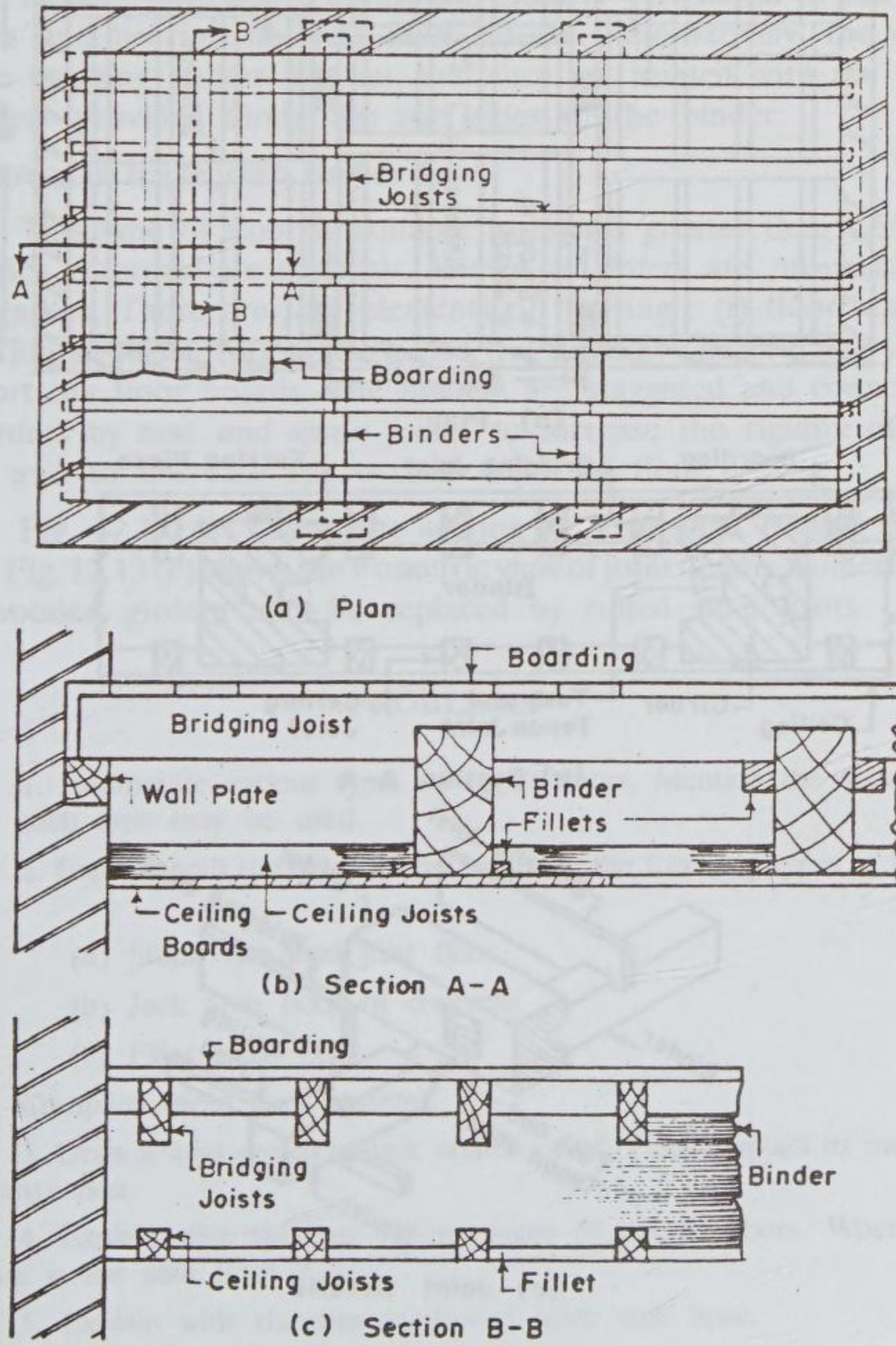
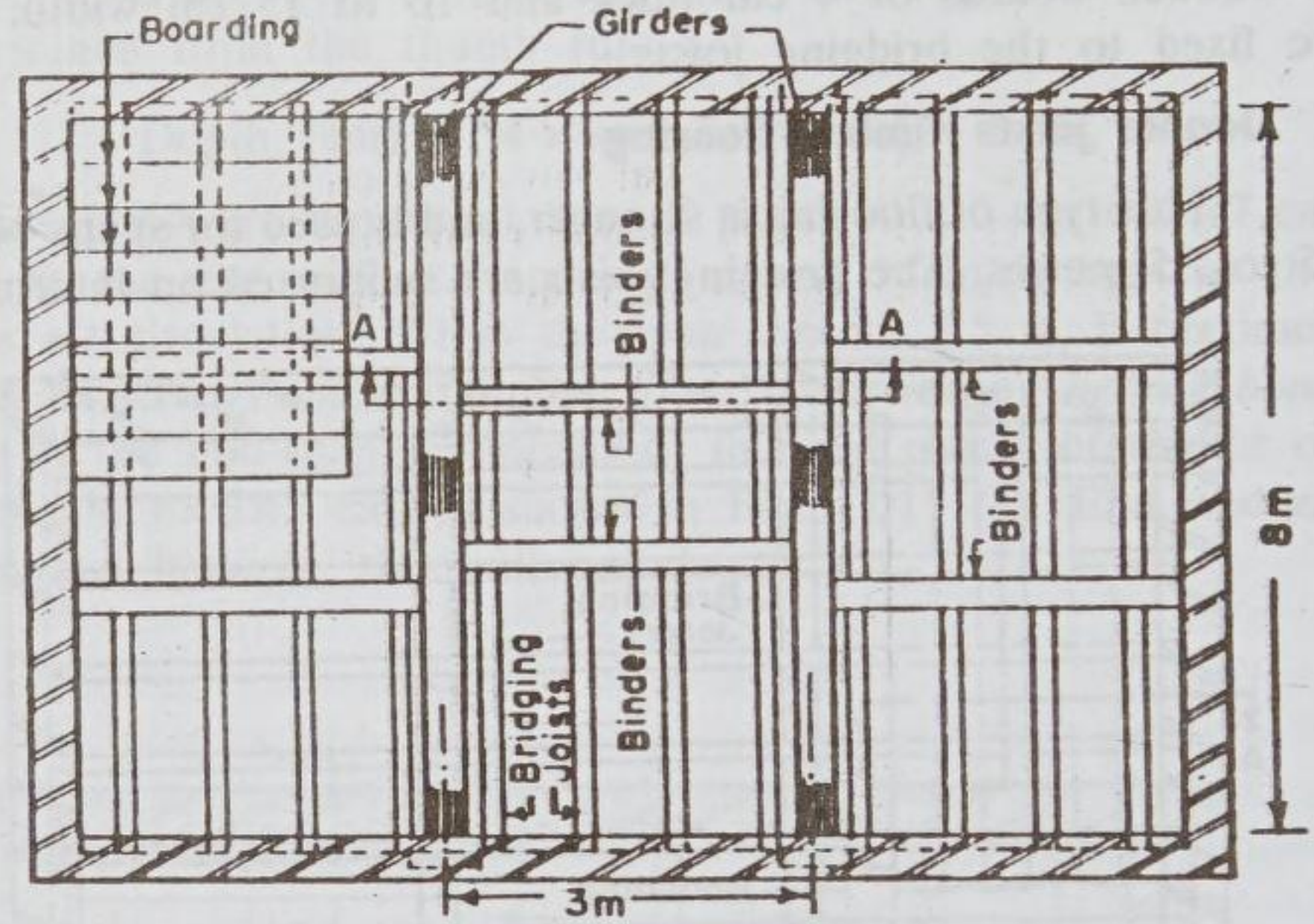
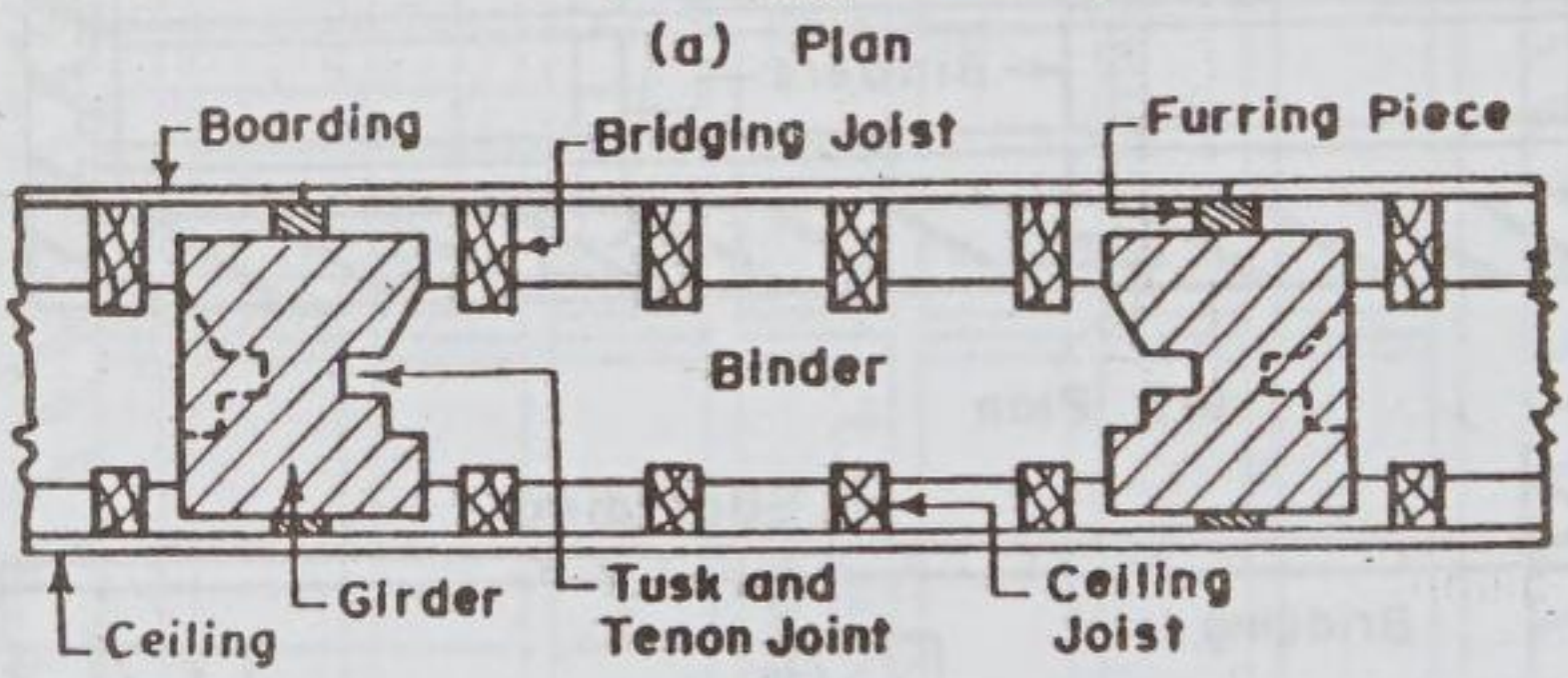


FIG. 12.12. DOUBLE JOISTS TIMBER FLOORING.

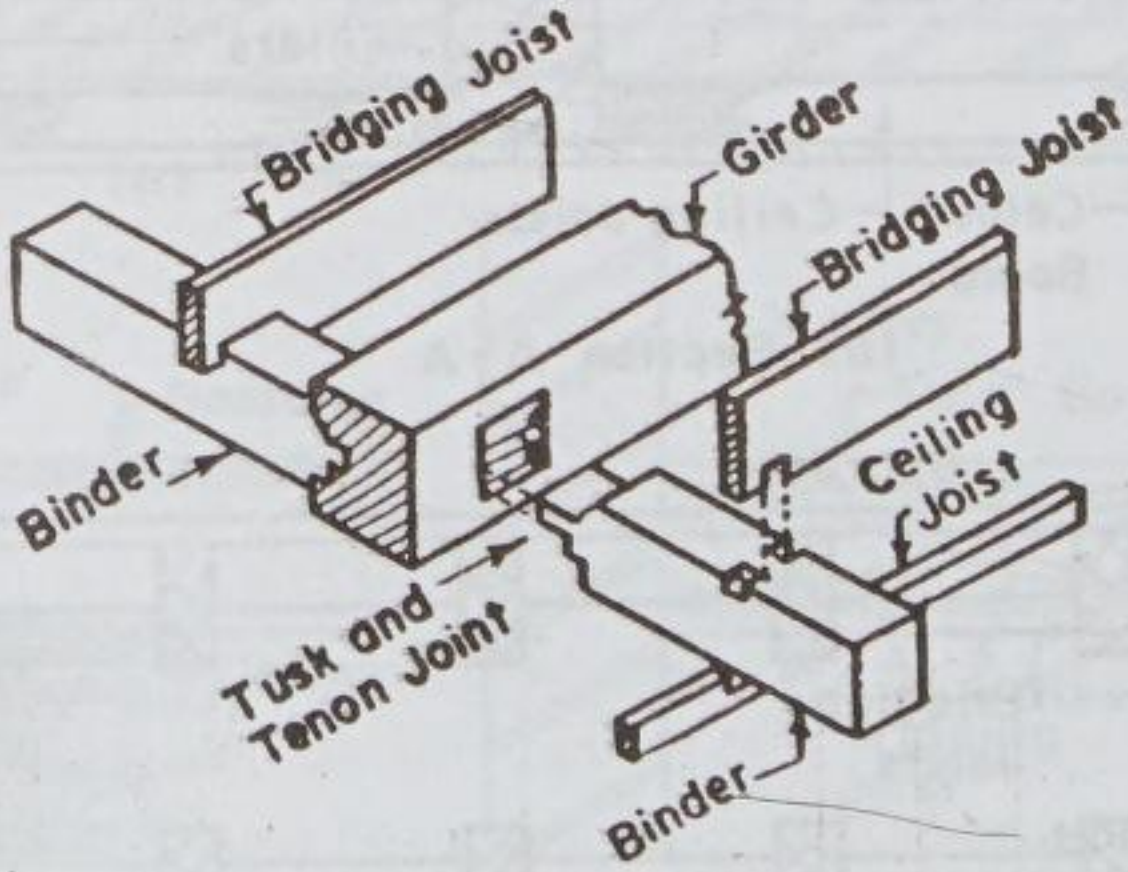
wooden supports, called *binders*. Thus, the loads of bridging joists are first transferred to the *binders* and through them to the end



(a) Plan



(b) Section A-A



(c) Joint Details

FIG. 12.13. FRAMED OR TRIPLE JOISTS TIMBER FLOORING.

walls in the form of highly concentrated loads. This is a disadvantage of this type of flooring. Also, the overall depth of the flooring is increased.

Because of intermediate supports, the bridging joists are of smaller sections, and are spaced at 30 cm centres. The spacing of binders is kept 2 to 3.5 m, and they rest on stone or wooden bearing templates which are not less than 0.75 to 2.5 m in length. In order to reduce the overall depth of the floor, bridging joists are cogged to the binders, with depth of sinking equal to $1/3^{rd}$ depth of bridging girders and bearing not less than 2.5 cm. Alternatively, the ends of the bridging girders are cut, and they are jointed with the help of fillers provided along the two sides of the binder.

3. Framed or triple joists timber floor

This type of floor is suitable for spans greater than 7.50 m, in which intermediate supports, known as *girders* are provided for the binders. There are *four* elements of flooring : (*i*) floor boards, (*ii*) bridging joists. (*iii*) binders, and (*iv*) girders. The bridging joists support the floor boards. The binders are staggered and connected to girders by *tusk and tenon joints,* to increase the rigidity of the floor and to decrease the overall depth of floor.

Fig. 12.13 (*b*) shows the section A-A through framed floor, while Fig. 12.13 (*c*) shows the isometric view of joint details. Sometimes, the wooden girders may be replaced by rolled steel joists.

PROBLEMS

1. Enumerate various types of upper floors. Mention the situations where each type may be used.

2. Explain, with the help of neat sketches, the following types of floors

(*a*) Stone slab-steel joist floor.

(*b*) Jack arch floor of concrete

(*c*) Filler joists floor.

Compare these floor systems.

3. Draw a neat sketch of jack arch floor of bricks. Explain its method of construction.

4. Explain with sketches various types of R.C.C. floors. Where do you use a flat slab floor ?

5. Explain with sketches reinforced brick slab floor.

6. What do you understand by ribbed floor ? Show the constructional details.

7. Write a note on precast concrete floor.

8. What are the different types of timber floors. Draw typical sketches of single and double joists timber floor.

9. Explain with the help of sketches, triple joists framed timber flooring.

13

Lintels and Arches

13.1. INTRODUCTION

Openings are invariably left in the wall for the provision of doors, windows, cupboards, almirahs, wardrobes, etc. These openings are bridged by the provision of either a lintel or an arch. Thus, both lintel as well as arch are structural members designed to support the loads of the portion of the wall situated above the openings, and then transmit the load to the adjacent wall portions (jambs) over which these are supported.

A *lintel* is a horizontal member which is placed across the opening. A lintel is thus a sort of beam, the width of which is equal to the width of the wall, and the ends of which are built into the wall. The bearing of lintel should be the minimum of the following :

(*i*) 10 cm.
(*ii*) Height of lintel.
(*iii*) 1/10th to 1/12th of the span of the lintel.

An arch is normally a curved member comprising of a mechanical arrangement of wedge shaped building units upholding each other by mutual pressure of their own weight and maintained in equilibrium by reaction from supports called *abutment*. However, arches of steel or reinforced concrete are built in single units of rigid nature, without the use of wedge shaped units . Bricks or masonry arches may also be flat.

Lintels are simple and easy to construct, while special centering/ from work is required for the construction of an arch. However, arches are constructed where (*i*) loads are heavy, (*ii*) span is more, (*iii*) strong abutment are available, and (*iv*) special architectural appearance is required.

13.2. CLASSIFICATION OF LINTELS

Lintels are classified into the following types, according to

the materials of their construction :

1. Timber lintels.
2. Stone lintels.
3. Brick lintels.
4. Steel lintels
5. Reinforced concrete lintels.

13.3. TIMBER LINTELS

Timber lintels are oldest types of lintels, though they are not commonly used now-a-days, except in hilly areas. Timber lintels are relatively costlier, structurally weak and vulnerable to fire. They are also liable to decay if not properly ventilated.

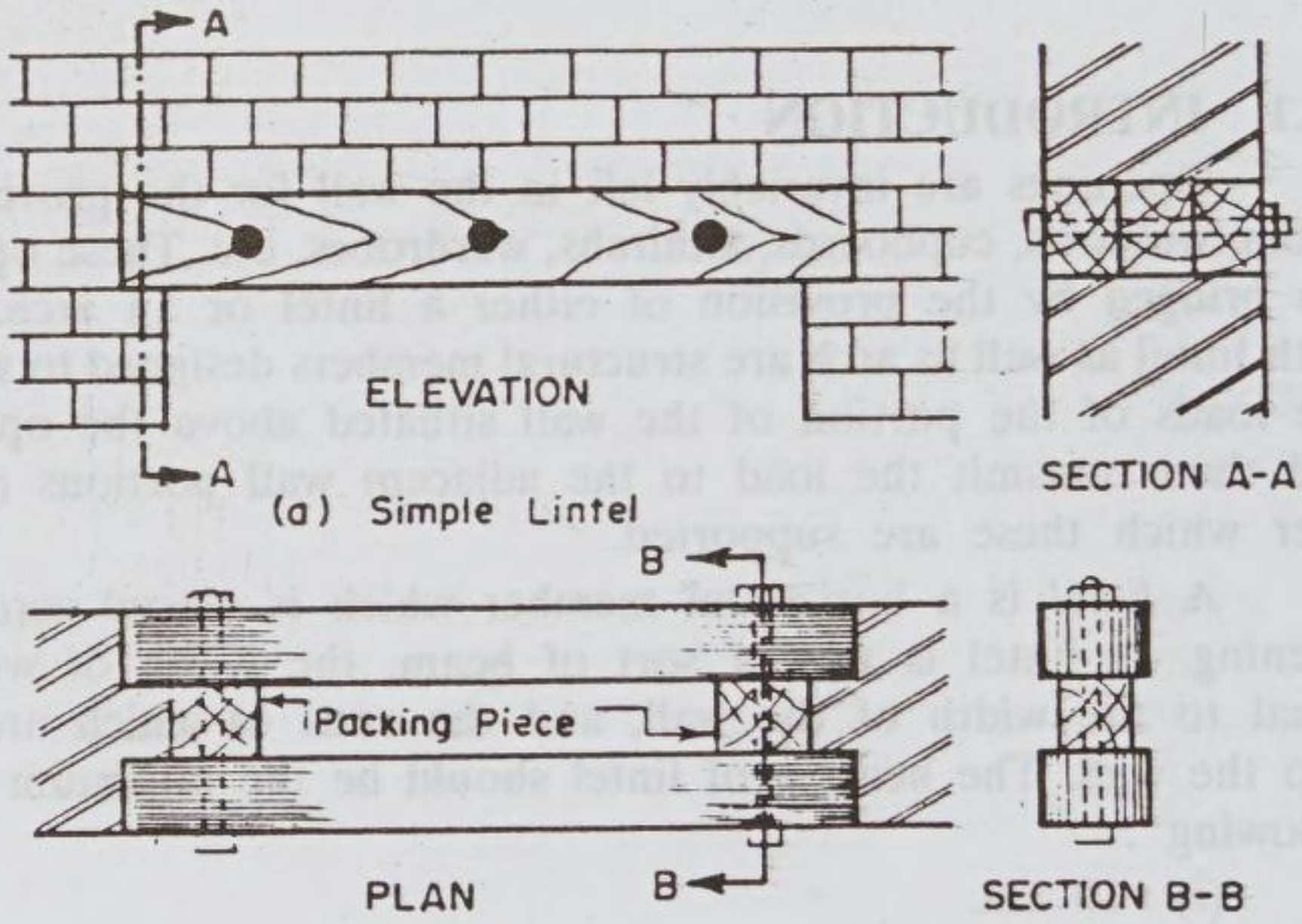


FIG. 13.1. WOODEN LINTEL.

Fig. 13.1 (*a*) shows a wooden lintel provided over the full width of the wall, by jointing together three timber pieces with the help of steel bolts. Fig. 13.1. (*b*) shows wooden lintel for a wider wall. The lintel is composed of two wooden pieces kept at a distance with the help of wooden distance pieces. Sometimes, timber lintels are strengthened by the provision of mild steel plates at their top and bottom, such lintels are called *flitched lintels.*

13.4. STONE LINTELS

Stone lintels are the most common types. Specially where stone is abundantly available. A stone lintel consists of a simple stone slab of greater thickness. Stone lintels can also be provided over openings in brick walls. Dressed stone lintels give good architectural appearance.

Stone lintels may be used in the form of either one piece or more than one piece along the width of the wall. The depth of stone lintel is kept equal to 10 cm per metre of span, with a minimum of 15 cm. They are used upto spans of 2 m. For wider spans, stone slabs are kept on edge.

Stone is very weak in tension. Also, it cracks if subjected to vibratory loads. Hence stone lintels should be used with caution where shock waves are quite common.

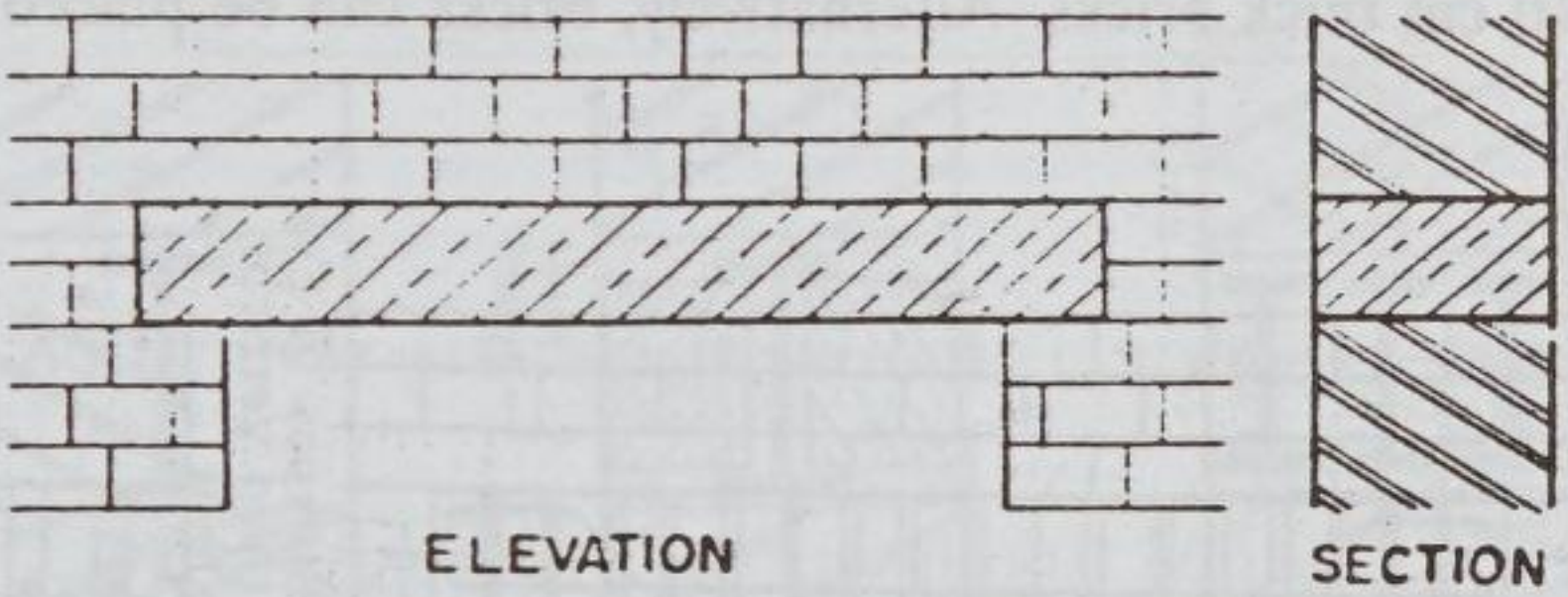


FIG. 13.2. STONE LINTEL.

13.5. BRICK LINTELS

Brick lintels are not structurally strong, and they are used only when the opening is small (less than 1 m) and loads are light. A brick lintel consists of bricks placed on end or edge, as shown in Fig. 13.3 (*a*). A better way of forming brick lintel is shown in Fig. 13.3 (*b*).

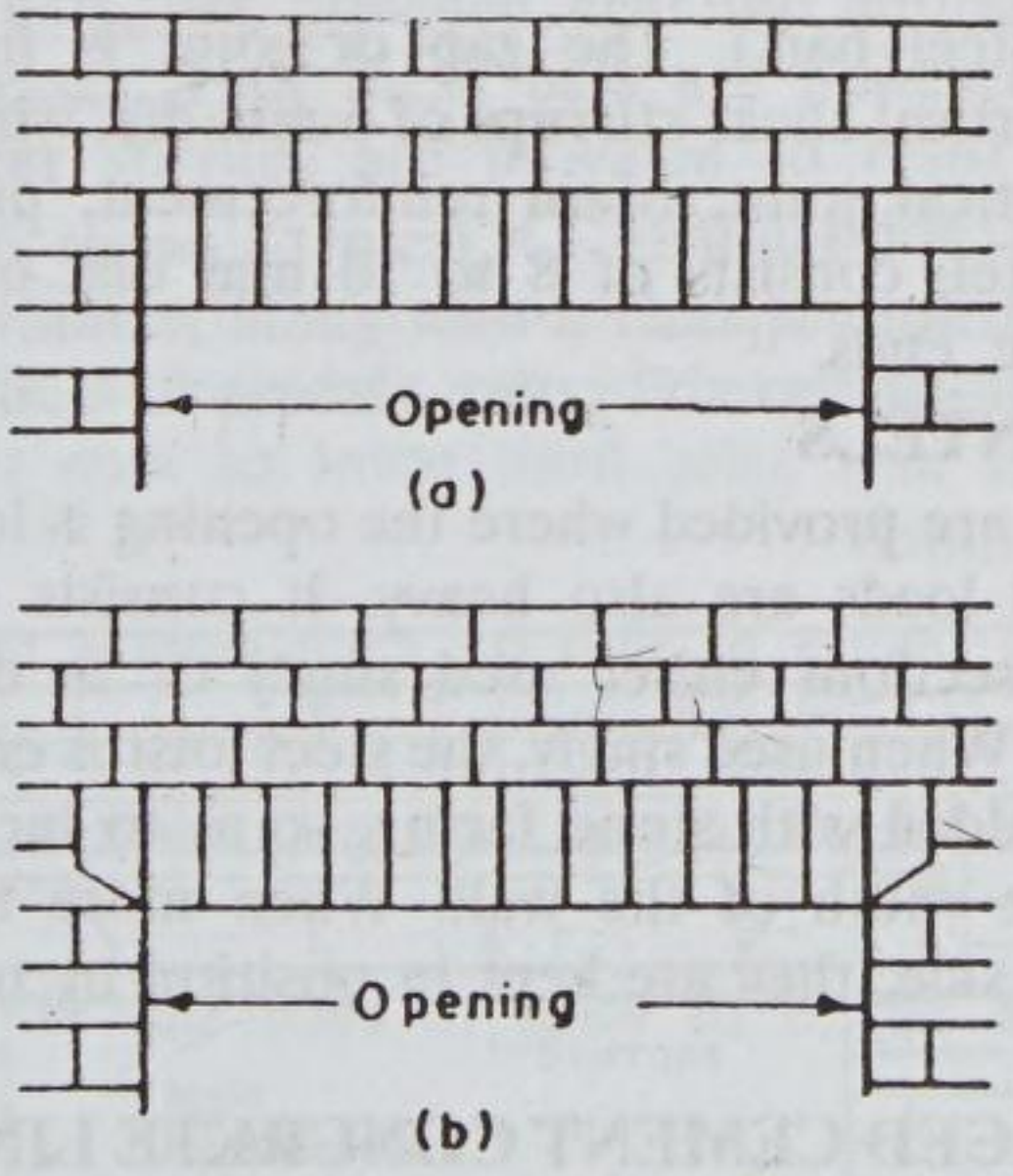


FIG. 13.3. BRICK LINTELS.

The depth of brick lintel varies from 10 to 20 cm, depending upon the span. It is constructed over temporary wooden centering. The bricks with frogs are more suitable for the construction of lintel

since the frogs, when filled with mortar, form joggles which increase the shear resistance of end joints. Such lintel is known *as joggled brick lintel.*

Reinforced Brick Lintel

Where loads are heavy, or span is more, lintels may be made of *reinforced brick work.* The depth of such lintel is kept equal to 10 cm, or in multiple of 10 cm. Sometimes, a 15 cm thick brick lintel may be obtained by using 5 cm thick tiles in conjunction with 10 cm thick bricks. Alternatively, bricks can be placed on edge.

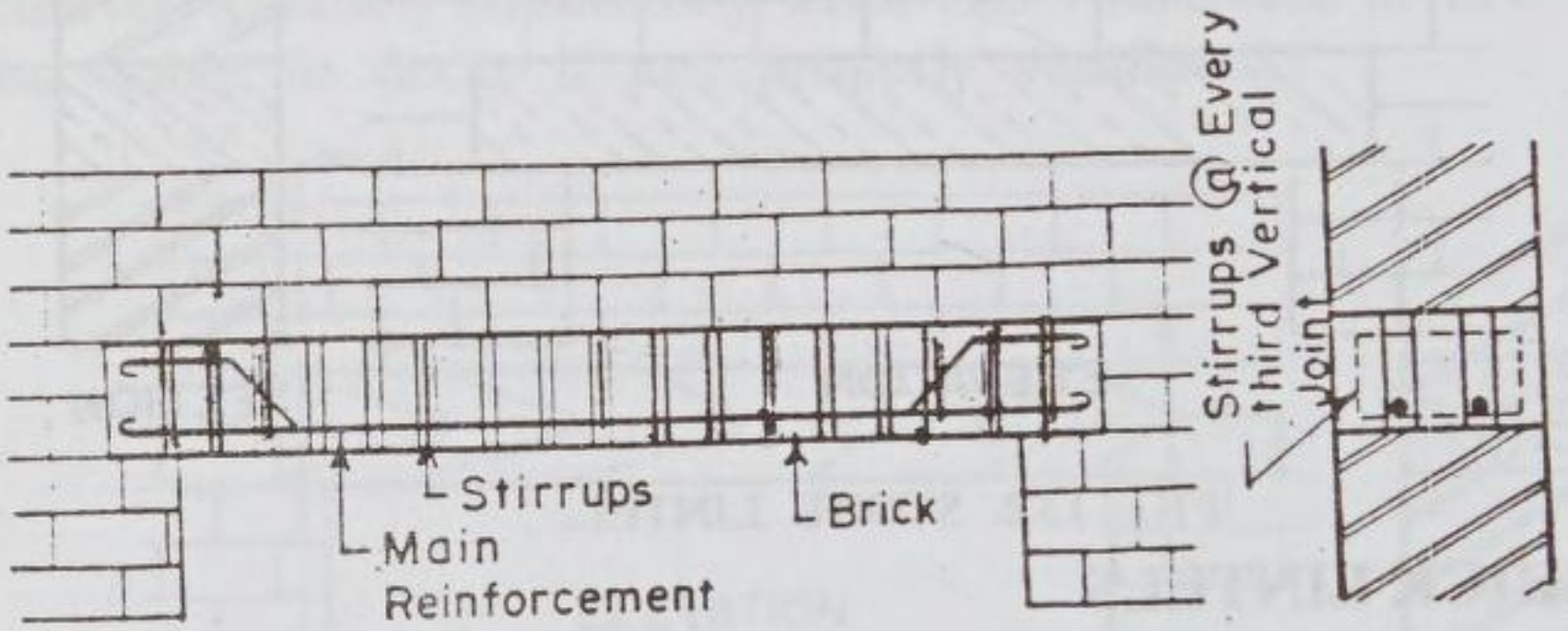


FIG. 13.4. REINFORCED BRICK LINTEL.

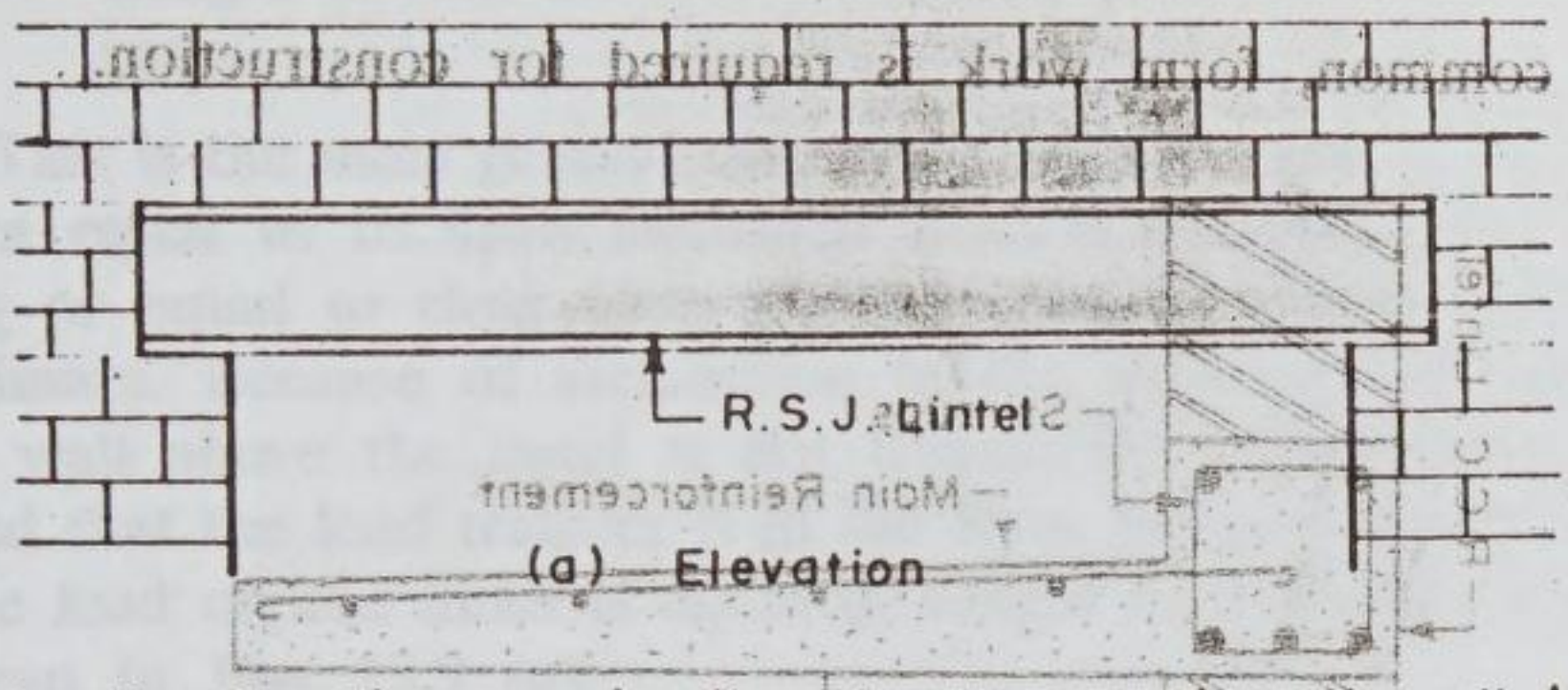
The bricks are so arranged that 2 to 3 cm wide space is left length-wise between adjacent bricks for the insertion of reinforcement (mild steel bars). The gap or joint is filled with 1 : 3 cement mortar. Vertical shear stirrups of 6 mm dia. wire are provided in every third vertical joint. Main reinforcement, provided at the bottom of the lintel, consists of 8 to 10 mm dia. bars, which are cranked up at the ends.

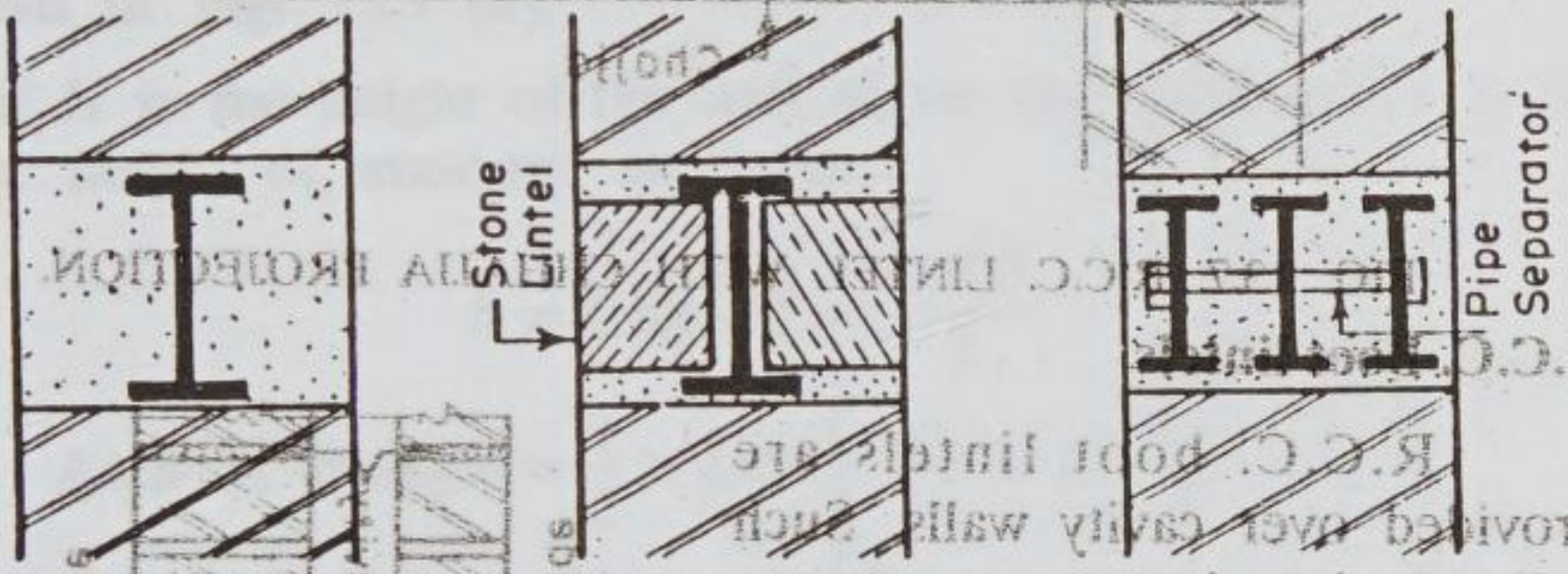
13.6. STEEL LINTELS

Steel lintels are provided where the opening is large and where the super-imposed loads are also heavy. It consists of rolled steel joists or channel sections either used singly or in combination of two or three units. When used singly, the steel joist is either embedded in concrete, or cladded with stone facing, so as to increase its width to match with the width of the wall. When more than one units are placed side by side, they are kept in position by tube separators. (Fig. 13.5)

13.7. REINFORCED CEMENT CONCRETE LINTELS

Reinforced cement concrete lintels have replaced practically all other types of lintels because of their strength, rigidity, fire resistance, economy and ease in construction. These can be used on any span. Its width is kept equal to the width of the wall. The





(b) Cross-section

FIG. 13.5. STEEL LINTELS.

depth of R.C.C. lintel and the reinforcement depends upon the span and the magnitude of loading. Longitudinal reinforcement, consisting of mild steel bars, are provided near the bottom of lintel to take up tensile stresses. Half these bars are however cranked up near the ends. Shear stirrups are provided to resist transverse shear.

Fig. 13.6. shows a typical R.C.C. lintel. Fig. 13.7 shows a R.C.C. lintel over a window, along with a chhajja projection. R.C.C. lintels are also available as precast units. For cast-in-situ units, which are

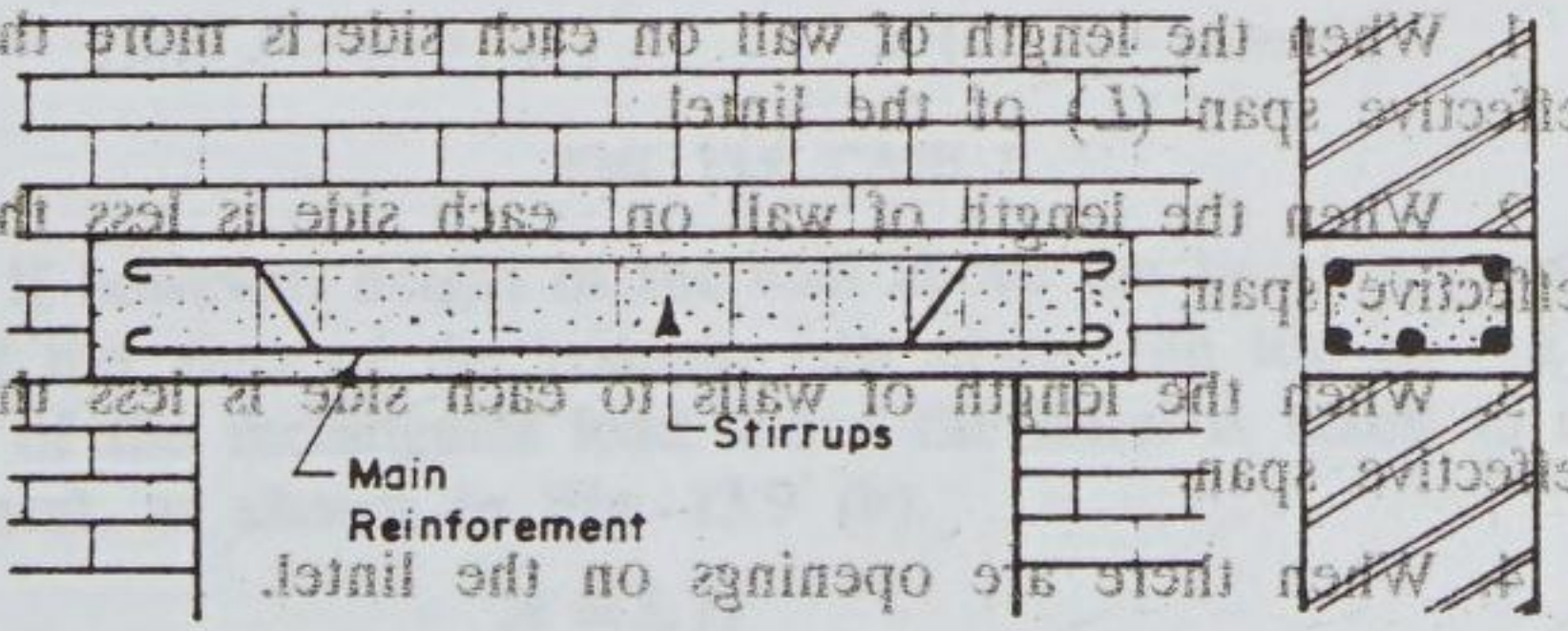


FIG. 13.6. R.C.C. LINTEL.

quite common, form work is required for construction.

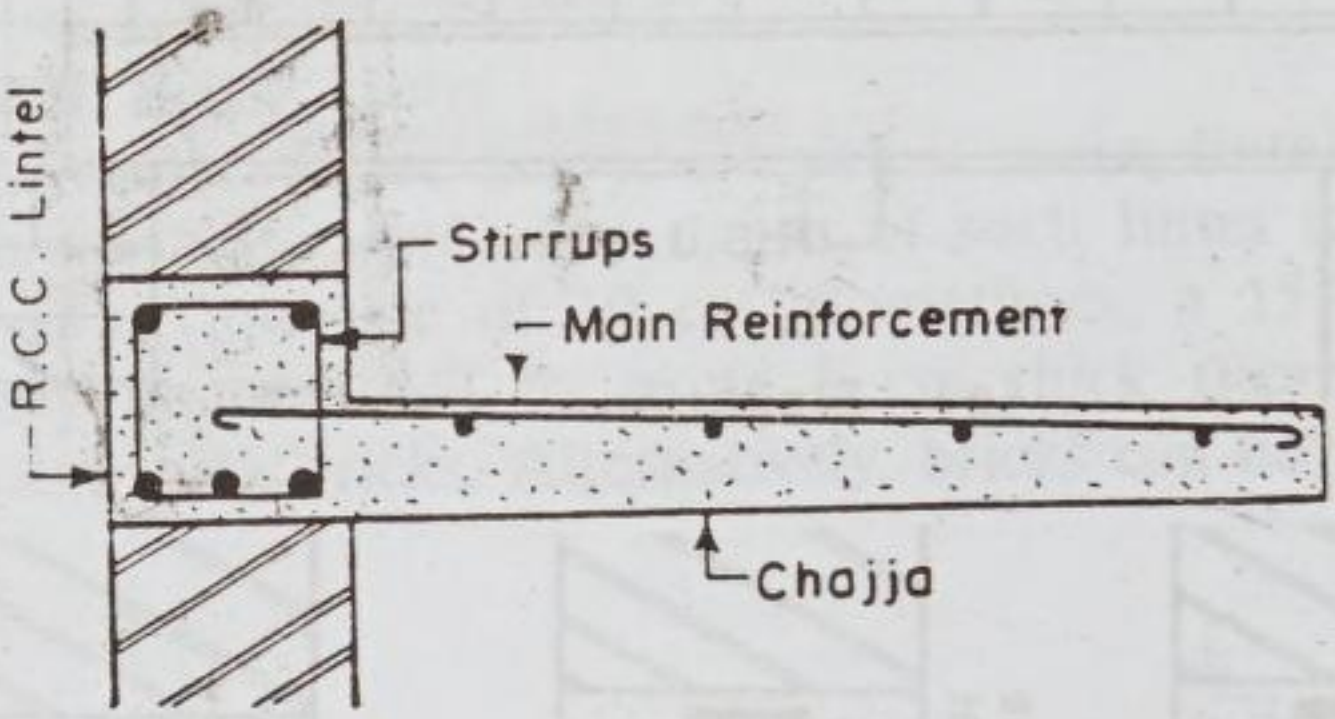


FIG. 13.7. R.C.C. LINTEL WITH CHHAJJA PROJECTION.

R.C.C. Boot lintels

R.C.C. boot lintels are provided over cavity walls. Such a lintel gives better appearance, and reduces quantity of concrete. However, the toe section of the boot lintel should be strong enough to sustain the loads. A flexible D.P.C. (damp-proof course) is provided above the lintel, as shown in Fig. 13.8.

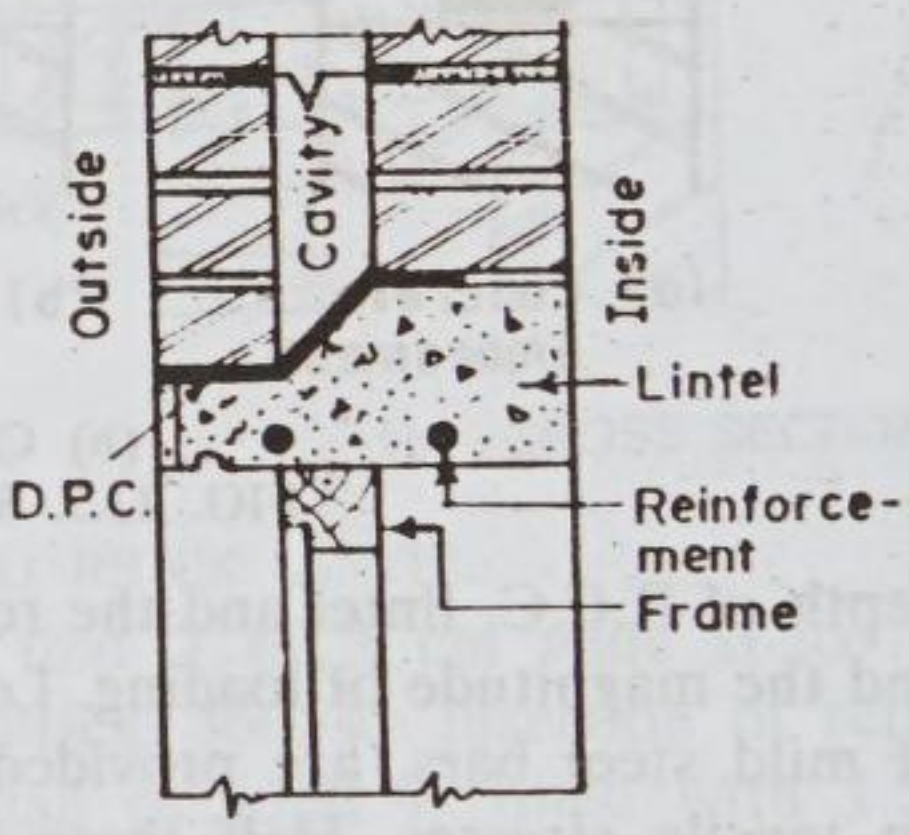


FIG. 13.8. R.C.C. BOOT LINTEL.

13.8. LOADING ON LINTELS

Lintels usually support the load of the wall over it and sometimes also the live load transferred by the slab-roof of the room. The following five cases may arise from point of view of distribution of load over the lintels :

1. When the length of wall on each side is more than half the effective span (L) of the lintel.

2. When the length of wall on each side is less than half the effective span.

3. When the length of walls to each side is less than half the effective span.

4. When there are openings on the lintel.

5. When there is load-carrying slab falling within dispersion triangle.

Case 1 : Length of wall on each side more than half the effective span

This is the most general case. The effective span of the lintel is taken equal to its span measured from centre to centre of its bearing or equal to clear span plus its effective depth, whichever is minimum. Because of arch-action in the masonry, all the load of the wall above the lintel is not transferred to the lintel. It is assumed that the load transfer is in the form of equilateral triangle, and the load on the lintel is equal to weight of triangular portion, as shown in Fig. 13.9 (*a*).

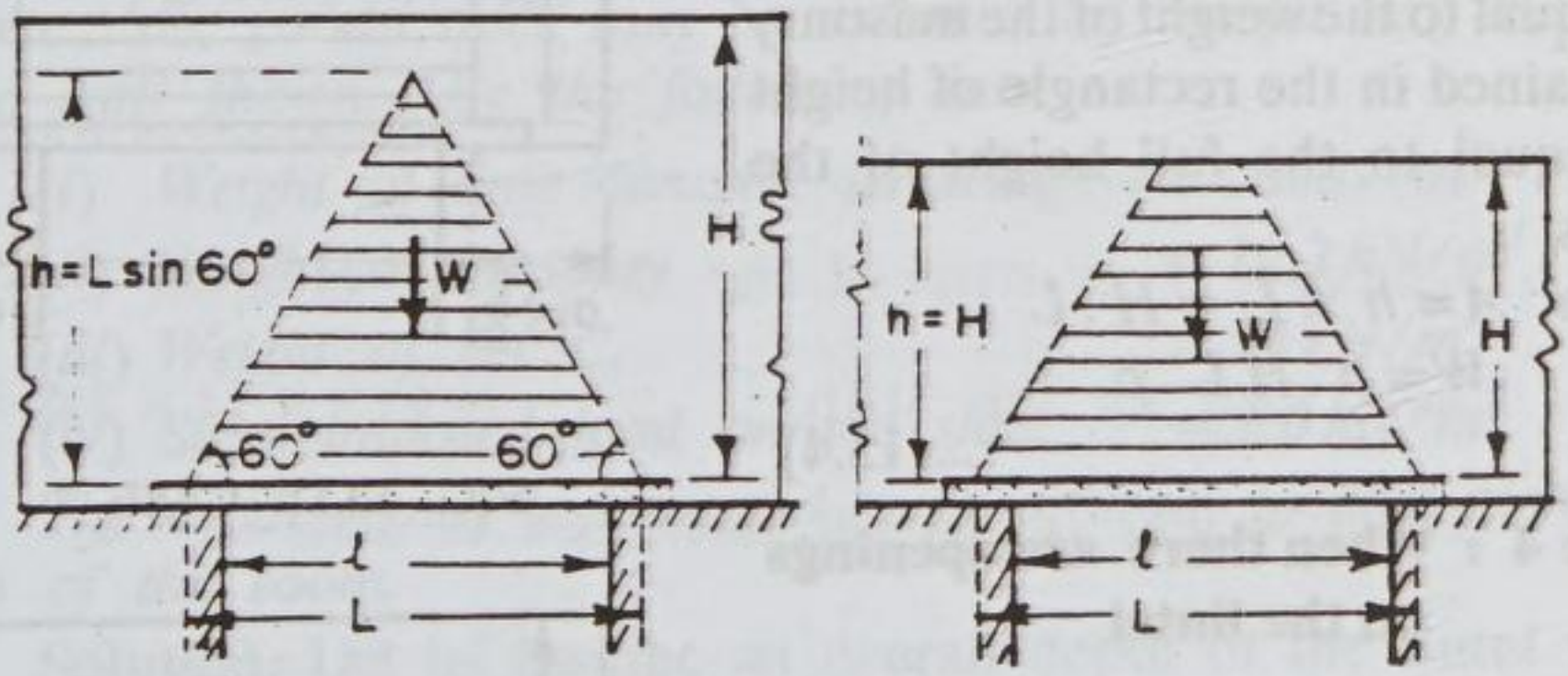
If H is the height of the wall above the lintel and h is the effective height of masonry, we have

$$h = L \sin 60° = \frac{L\sqrt{3}}{2}$$

$$\therefore \text{ Area of triangle} = \frac{1}{2} \times L \times \frac{L}{2}\sqrt{3} = L^2 \frac{\sqrt{3}}{4}$$

$$\therefore \qquad \text{Load } W = b\, L^2 \rho \frac{\sqrt{3}}{4} \qquad \ldots(13.1)$$

where b = width of wall ; ρ = unit weight(kg/m^3 or kN/m^3) of masonry.



(a) $H > L \sin 60°$ (b) $H < L \sin 60°$

FIG. 13.9. CASE 1.

If however, height of the wall above the lintel is insufficient (*i.e.*, if the apex of the triangle falls above the top of the wall), whole of the rectangular load above the lintel is taken to act on the lintel, as shown in Fig. 13.9 (*b*).

$$A = L\,H$$

and

$$W = b\,L\,H\,\rho \qquad \ldots(13.2)$$

Case 2 : Length of the wall on one side less than half the effective span

Fig. 13.10 shows the situation where the length of wall to one side is less than half the effective span (*i.e.* $a_1 < L/2$) but the length to the other side is more than half the effective span ($a_2 > L/2$). In that case, the load transferred to the lintel will be equal to the weight of masonry contained in the rectangle of the height h equal to the effective span.

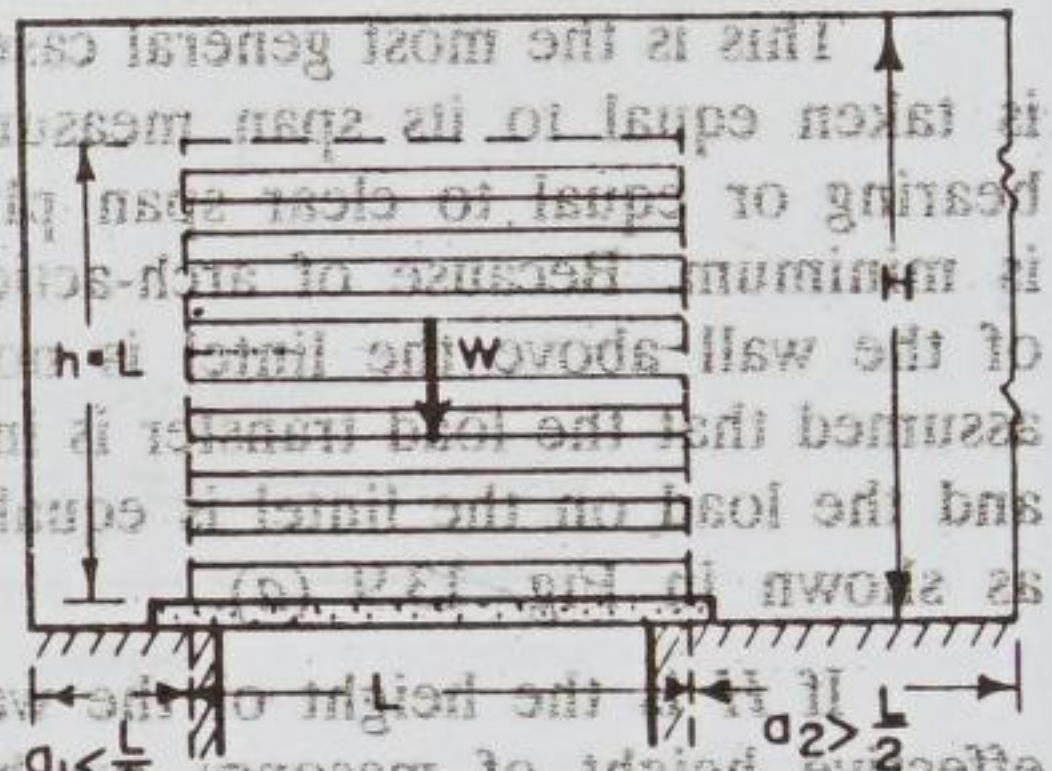


FIG. 13.10. CASE 2.

Thus, $A = h \times L = L \times L = L^2$

and $W = b\,L^2\rho$ ...(13.3)

Case 3 : Length of the walls to each side less than half the effective span

This is shown in Fig. 13.11. The load acting on the lintel will be equal to the weight of the masonry contained in the rectangle of height h equal to the full height of the walls.

Thus, $A = h \times L = H.L$

and $W = b.HL.\rho$ ...(13.4)

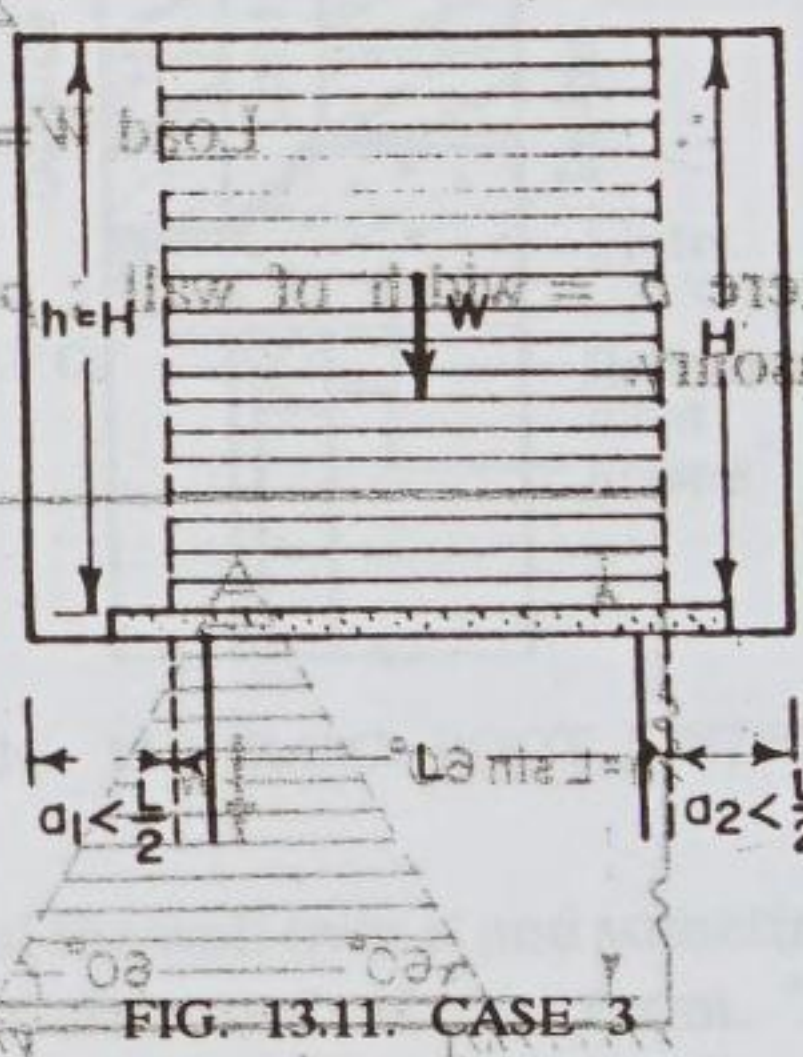


FIG. 13.11. CASE 3

Case 4 : When there are openings on the lintel

If there are openings, due to the provision of ventilators etc., and if these openings are intersected by the 60° lines, the loading will be calculated by allowing dispersion lines at 60° from the top edges of the openings, as shown in Fig. 13.12. The total load on the lintel will be equal to the weight of the masonry contained in the shaded area.

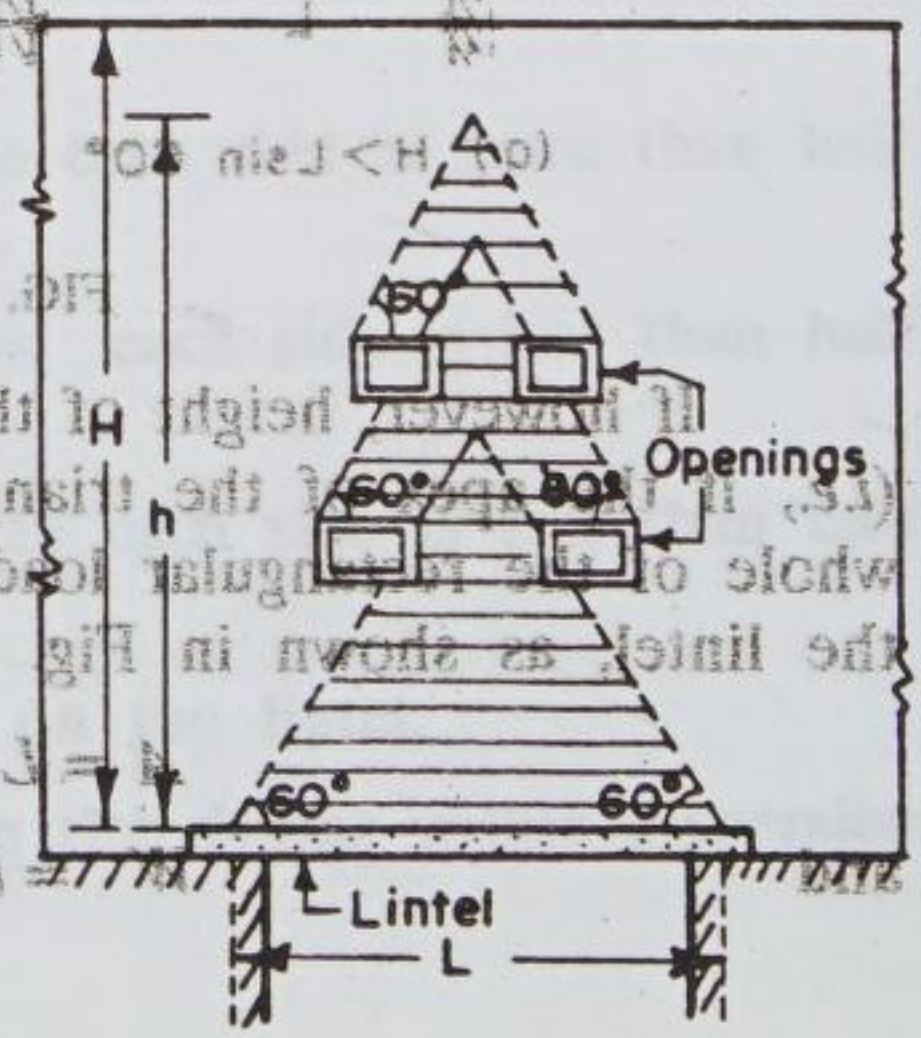


FIG. 13.12 CASE 4 : OPENINGS.

Case 5 : Load-carrying slab falling within the dispersion triangle

If the roof slab is provided at a level well above the apex of the dispersion triangle, the uniformly distributed load carried by the slab is *not* transferred to the lintel. If, however, the slab intersects the dispersion triangle (Fig. 13.13), three types of loads are transferred to the lintel :

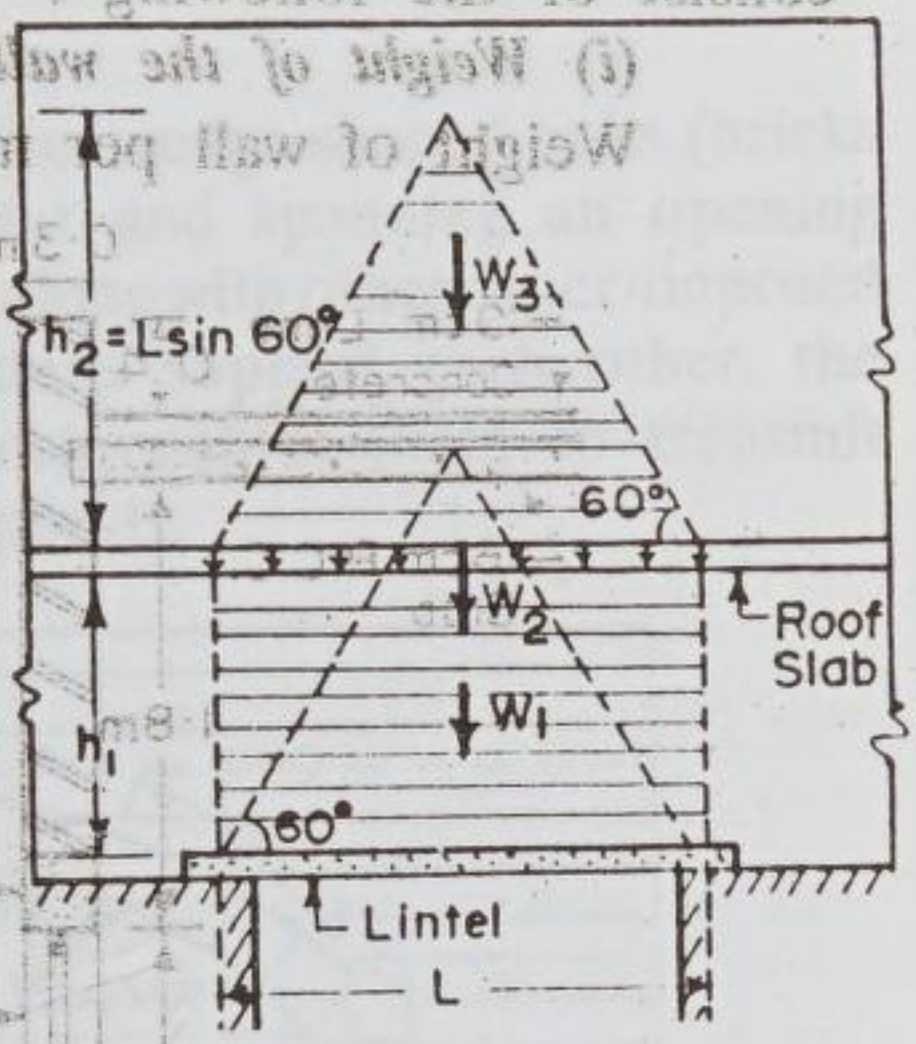


FIG. 13.13. CASE 5 : LOAD FROM SLAB.

(*i*) Load W_1 due to the weight of the masonry contained in the rectangle of height h_1 equal to the height of the slab above the lintel.

(*ii*) Load W_2 carried by the slab, in a length L.

(*iii*) Load W_3 due to weight of the masonry contained in the equilateral triangle above the slab, where the height h_3 of the triangle is equal L sin 60°.

Example. 13.1. *Fig. 13.14 shows the cross-section of a wall of a room, 5 m wide and 18 m long from inside. Find the load transferred to the lintel for the window having a clear opening of 2 m. The reinforced concrete slab, 16 cm thick may be assumed to transfer half the load to the wall shown. Use the following data :*

(*i*) *Weight of lime concrete terracing* $= 19\ kN/m^3$

(*ii*) *Weight of masonry* $= 19.2\ kN/m^3$

(*iii*) *Weight of R.C.C.* $= 24\ kN/m^3$

(*iv*) *Super-imposed load on the slab* $= 1.6\ kN/m^2$

The window may be assumed to be centrally located along the length of the room.

Solution. Let us assume an overall depth of the lintel to be 30 cm and a bearing of 30 cm on each side. Hence effective span of lintel will be equal to 2 + 0.3 = 2.3 m. Assuming the dispersion to be along 60° lines, the height of load triangle, measured above the top of the lintel $= 2.3 \sin 60° = \dfrac{2.3 \times \sqrt{3}}{2} \approx 2$ m, which is more than height 1.5 m of the slab above the lintel. Hence the load of the slab will be transferred to the lintel.

Fig. 13.14 (*o*) shows the elevation, showing all the heights. If we construct equilateral triangle above the top of the slab, its apex will fall very much above the top of the parapet wall. Hence the weight of the whole wall above the lintel will be transferred to the lintel. Thus, the loads per metre length of the lintel will

consist of the following :

(i) Weight of the wall

Weight of wall per metre run $= 0.3 \times 2.16 \times 19 = 12.31$ kN

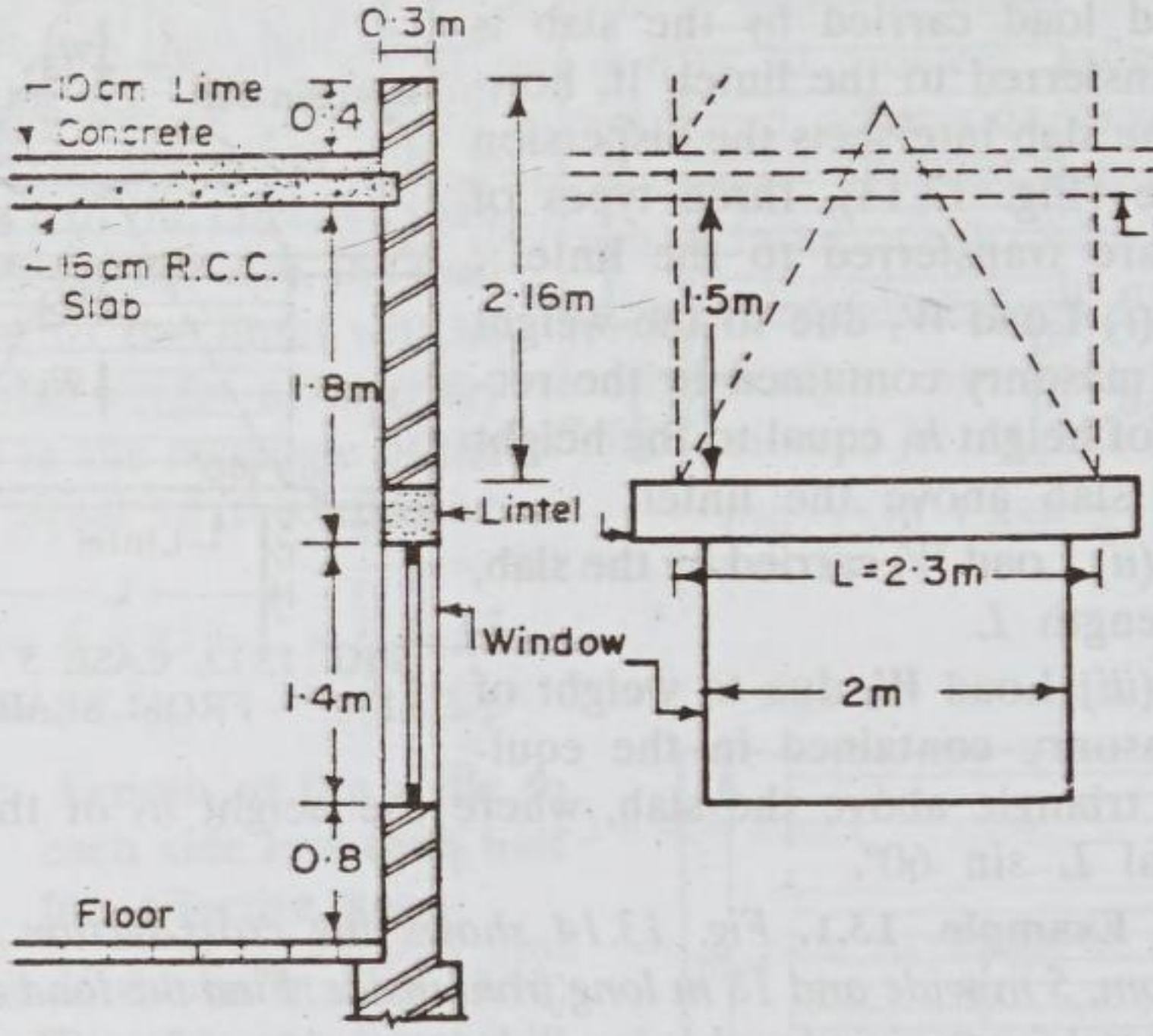


FIG 13.14

(ii) Load transferred by the slab

Consider 1 m strip of the slab, of 5 m span

Live load on this strip $= \frac{1}{2}(5 \times 1) \times 1.6 = 4$ kN

Dead weight of 16 cm thick R.C.C. slab

$$= \frac{1}{2}\left(5 \times 1 \times \frac{16}{100}\right) 24 = 9.6 \text{ kN}$$

Dead weight of lime concrete

$$= \frac{1}{2}\left(5 \times 1 \times \frac{10}{100}\right) 19 = 4.75 \text{ kN.}$$

Total load/m transferred from slab $= 18.35$ kN

(iii) Self weight of lintel

Weight per metre run $= 0.3 \times 0.3 \times 1 \times 24 = 2.16$ kN

Hence total load per metre run of lintel

$$= 12.31 + 18.35 + 2.16 \text{ kN/m} = 32.82 \text{ kN/m}$$

Total load on lintel $= 32.82 \times 2.3 \approx 75.5$ kN

The structural design of the lintel has to be done for this loading.

13.9. ARCH : TERMS USED

An arch is a structure constructed of wedge-shaped units (bricks or stone), jointed together with mortar and spanning an opening to support the weight of the wall above it along with other super-imposed loads. Due to wedge-like form, the units support each other, the load tends to make them compact and enables them to transmit the pressure downwards to their supports.

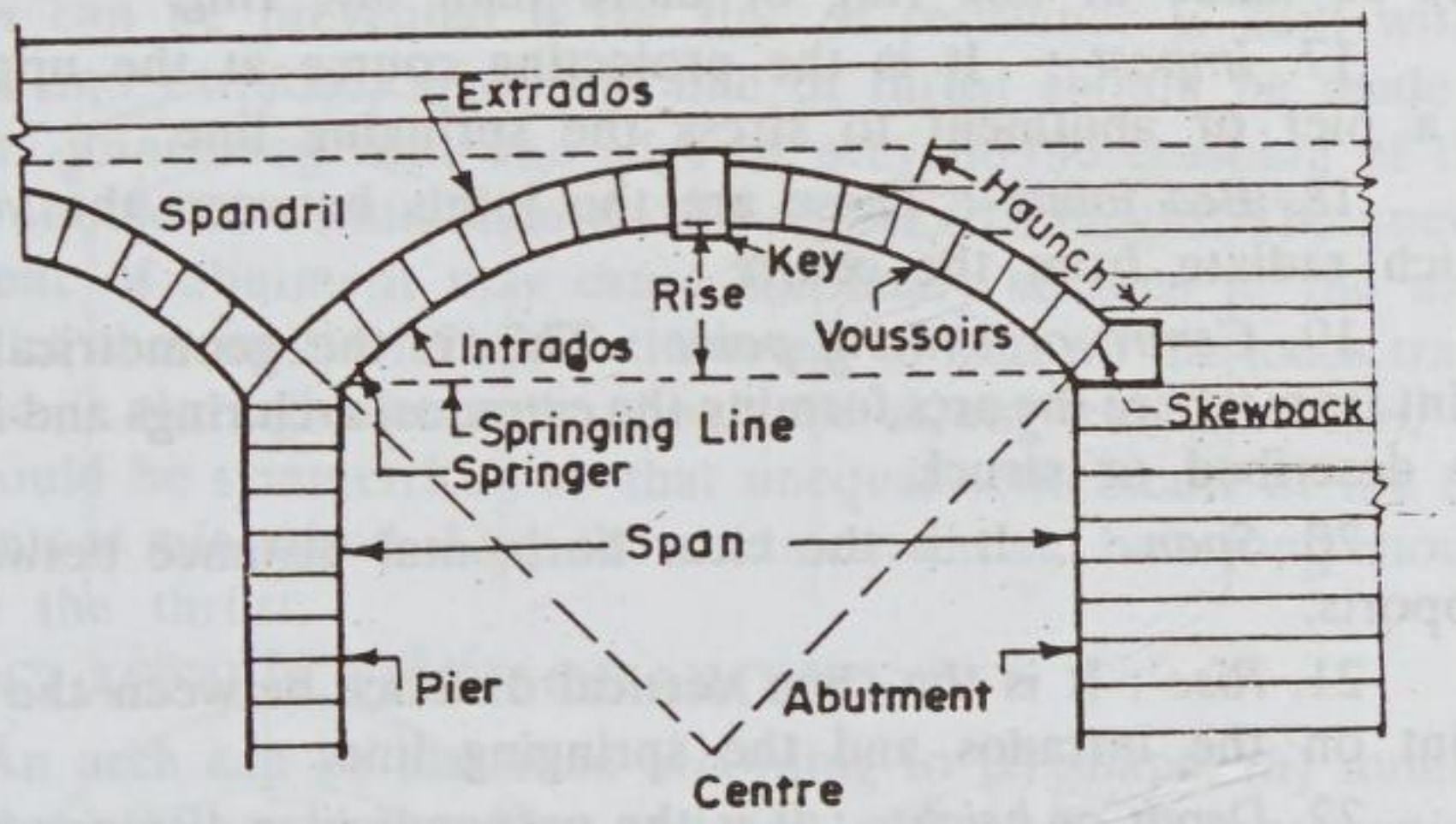


FIG 13.15. ELEMENTS OF A SEGMENTAL ARCH

Fig. 13.15 shows varicus elements of an arch. The following technical terms are used in arch work :

1. *Intrados* : This is the inner curve of an arch.

2. *Soffit* : It is the inner surface of an arch. Sometimes, intrados and soffit are used synonymously.

3. *Extrados* : It is the outer curve of an arch.

4. *Voussoirs* : These are wedge-shaped units of masonry, forming an arch.

5. *Crown* : It is the highest part of extrados.

6. *Key* : It is the wedge-shaped unit fixed at the crown of the arch.

7. *Spandril :* This is a curved-triangular space formed between the extrados and the horizontal line through the crown.

8. *Skew back* : This is the inclined or splayed surface on the abutment, which is so prepared to receive the arch and from which the arch springs.

9. *Springing points* : These are the points from which the curve of the arch springs.

10. *Springing line* : It is an imaginary line joining the springing points of either end.

11. *Springer* : It is the first voussoir at springing level; it is immediately adjacent to the skewback.

12. *Abutment* : This is the end support of an arch.

13. *Pier* : This is an intermediate support of an arcade.

14. *Arcade* : It is a row of arches in continuation.

15. *Haunch* : It is the lower half of the arch between the crown and skew back.

16. *Ring* : It is a circular course forming an arch. An arch may be made of one ring or more than one ring.

17. *impost* : It is the projecting course at the upper part of a pier or abutment to stress the springing line.

18. *Bed joints* : These are the joints between the voussoirs which radiate from the centre.

19. *Centre or striking point* : This is the geometrical centre point from where the arcs forming the extrados, arch rings and intrados are described or struck.

20. *Span* : It is the clear horizontal distance between the supports.

21. *Rise* : It is the clear vertical distance between the highest point on the intrados and the springing line.

22. *Depth or height* : It is the perpendicular distance between the intrados and extrados.

23. *Thickness (or breadth of soffit)* : This is the horizontal distance, measured perpendicular to the front and back faces of an arch.

13.10. STABILITY OF AN ARCH

An arch transmits the super-imposed load to the side walls (or abutments) through friction between the surfaces of voussoirs and the cohesion of mortar. Every element of arch remains in compression. It has also to bear transverse shear. An arch may therefore fail in the following ways

(*i*) Crushing of the masonry

(*ii*) Sliding of voussoir

(*iii*) Rotation of some joint about an edge

and (*iv*) Uneven settlement of abutment/pier.

If the compressive stress or thrust exceeds the safe crushing strength of the materials (*i.e.*, masonry units and mortar), the arch will fail in crushing. Hence, the material used for construction should be of adequate strength, and the size of voussoirs should be properly designed to bear the thrust transmitted through them. The height of voussoirs should not be less than 1/12th the span. For span upto 1.5 m, 20 cm thick arch ring is provided, while for span between

1.5 to 4 m, 30 cm thickness is sufficient. For span between 4 to 6.5 m, 40 cm thickness should be provided while for span more than 6.5 m, the thickness at springing may be increased by about 20% of the thickness at the crown. Sometimes, voussoirs of variable heights are provided–less height near crown and more height at skew-back. To safeguard against sliding of voussoirs past each other due to transverse shear, the voussoirs of greater height should be provided. Also, the angle between the line of resistance of the arch and the normal to any point should be less than angle of internal friction. *Rotation* can be prevented if the line of resistance is kept within intrados and extrados. Also, the line of thrust should be made to cross the joint away from the edge to prevent the crushing of that edge. It should be within middle third of the arch height. The uneven settlement of abutment may cause secondary stresses in the arch. Hence the abutment, which has ultimately to bear all the loads transferred to it through the arch, should be strong enough. Also, the arch should be symmetrical, so that unequal settlements of the two abutments is minimised. Also, the abutment should be strong enough to take the thrust.

13.11 . CLASSIFICATION OF ARCHES

An arch can be classified according to (*i*) shape, (*ii*) number of centres, (*iii*) workmanship, and (*iv*) materials of construction.

(a) Classification according to shape

According to this classification, arches may be of the following types : (*i*) Flat arch (*ii*) Segmental arch (*iii*) Semi-circular arch (*iv*) Horse shoe arch (*v*) Pointed arch or gothic arch (*vi*) Venetian arch (*vii*) Florentine arch (*viii*) Relieving arch (*x*) Stilled arch, and (*xi*) Semi-elliptical arch.

1. Flat Arch (Fig. 13.16 *a*)

A flat arch has usually the angle formed by skewbacks as 60° with horizontal, thus forming an equilateral triangle with intrados as the base. The intrados is apparently flat, but it is given a slight rise of camber of about 10 to 15 mm per metre width of opening to allow for small settlements. However, the extrados is kept horizontal and flat. Flat arches are used only for light loads, and for spans upto 1.5 m.

2. Segmental Arch (Figs. 13.15 and 13.16 *b*)

This is the common type of arch used for buildings. The centre of arch lies below the springing line. The thrust transferred to the abutment is in an inclined direction.

3. Semi-circular Arch (Fig. 13.16 *c*)

This is the modification of segmental arch in which the centre lies on the springing line. The shape of the arch curve is that of a semi-circle. The thrust transferred to the abutments is perfectly

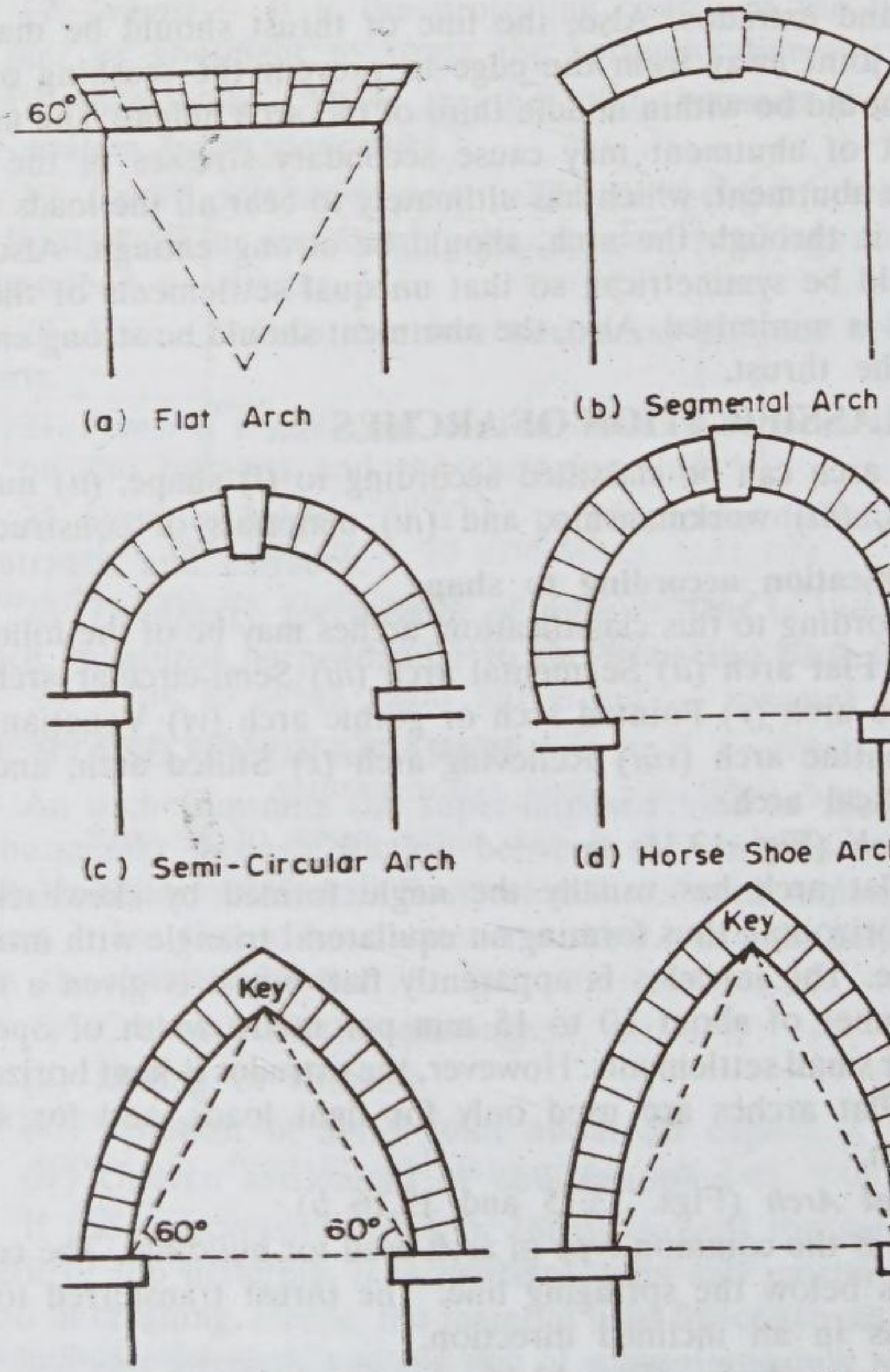
in vertical direction since the skewback is horizontal.

4. Horse Shoe Arch (Fig. 13.16 *d*)

The arch has the shape of a horse shoe, incorporating more than a semi-circle. Such type of arch is provided mainly from architectural considerations.

5. Pointed Arch (Fig. 13.16 *e*)

This is also known as Gothic arch. It consists two arcs of circles meeting at the apex. The triangle formed may be *equilateral* or isosceles; in the latter case it is known as Lancet arch.



(a) Flat Arch

(b) Segmental Arch

(c) Semi-Circular Arch

(d) Horse Shoe Arch

(i) Equilateral Arch

(ii) Isosceles Arch

(e) Pointed Arches

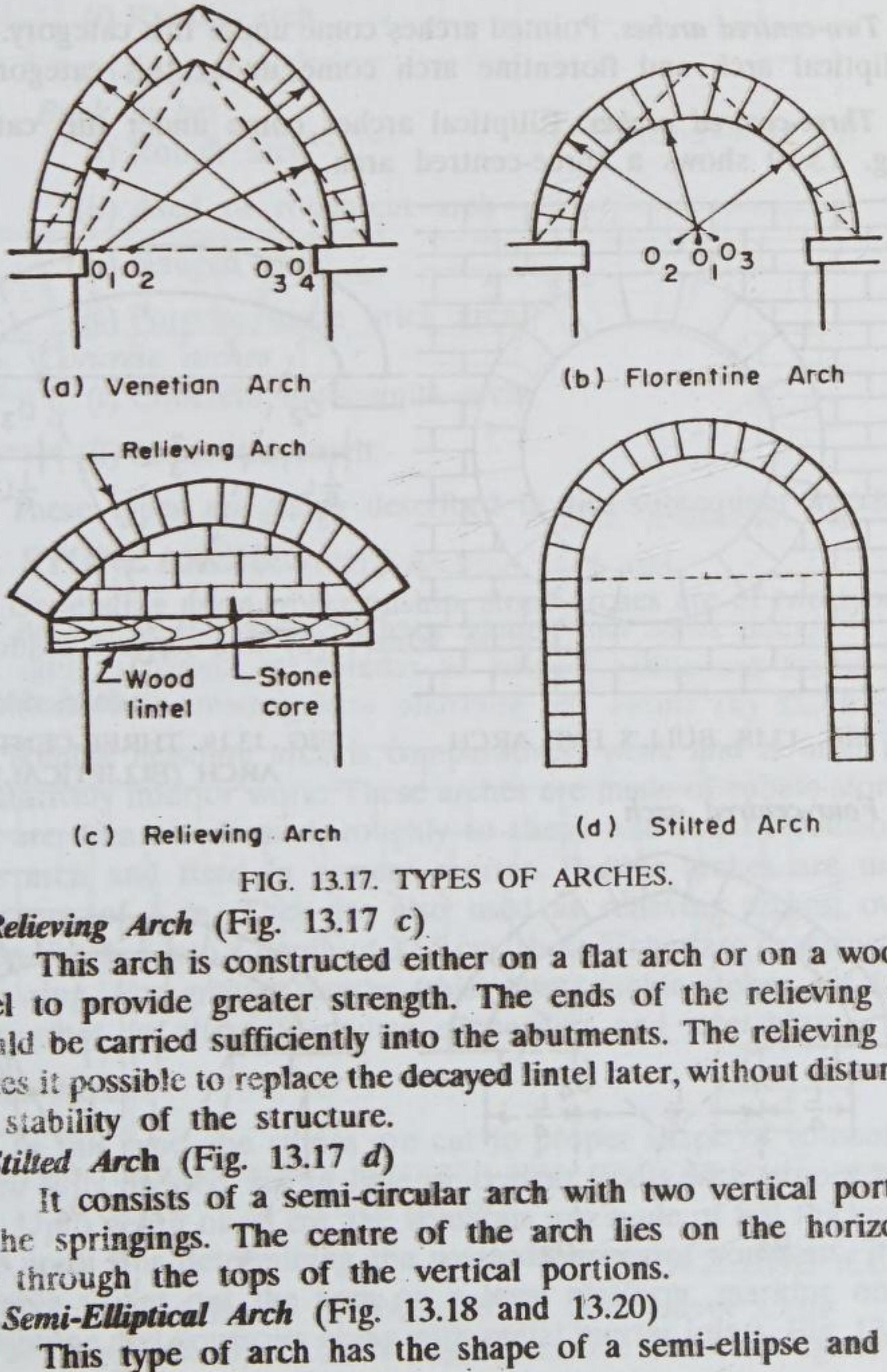
FIG. 13.16. TYPES OF ARCHES.

6. Venetian Arch (Fig. 13.17 *a*)

This is another form of pointed arch which has deeper depth at crown than at springings. It has four centres, all located on the springing line.

7. Florentine Arch (Fig. 13.17 *b*)

This is similar to Venetian arch except that the intrados is a semicircle. The arch has, thus three centres, all located on the springing line.



(a) Venetian Arch (b) Florentine Arch

(c) Relieving Arch (d) Stilted Arch

FIG. 13.17. TYPES OF ARCHES.

8. Relieving Arch (Fig. 13.17 *c*)

This arch is constructed either on a flat arch or on a wooden lintel to provide greater strength. The ends of the relieving arch should be carried sufficiently into the abutments. The relieving arch makes it possible to replace the decayed lintel later, without disturbing the stability of the structure.

9. Stilted Arch (Fig. 13.17 *d*)

It consists of a semi-circular arch with two vertical portions at the springings. The centre of the arch lies on the horizontal line through the tops of the vertical portions.

10. Semi-Elliptical Arch (Fig. 13.18 and 13.20)

This type of arch has the shape of a semi-ellipse and may have either three centres or five centres.

(*b*) Classification based on number of centres

The arches may be classified as (*i*) one-centred arch (*ii*) two-centred arch (*iii*) three-centred arch (*iv*) four-centred arch and (*v*) five-centred arch.

*1. **One-centred arches.*** Segmental arches, semicircular arches, flat arches, horse-shoe arch and stilted arches come under this category. Sometimes, a perfectly circular arch, known as *bull's eye arch* is provided for circular windows, as shown in Fig. 13.18.

*2. **Two-centred arches.*** Pointed arches come under this category. Semi-elliptical arch and florentine arch come under this category.

*3. **Three-centred arches.*** Elliptical arches come under this category. Fig. 13.19 shows a three-centred arch.

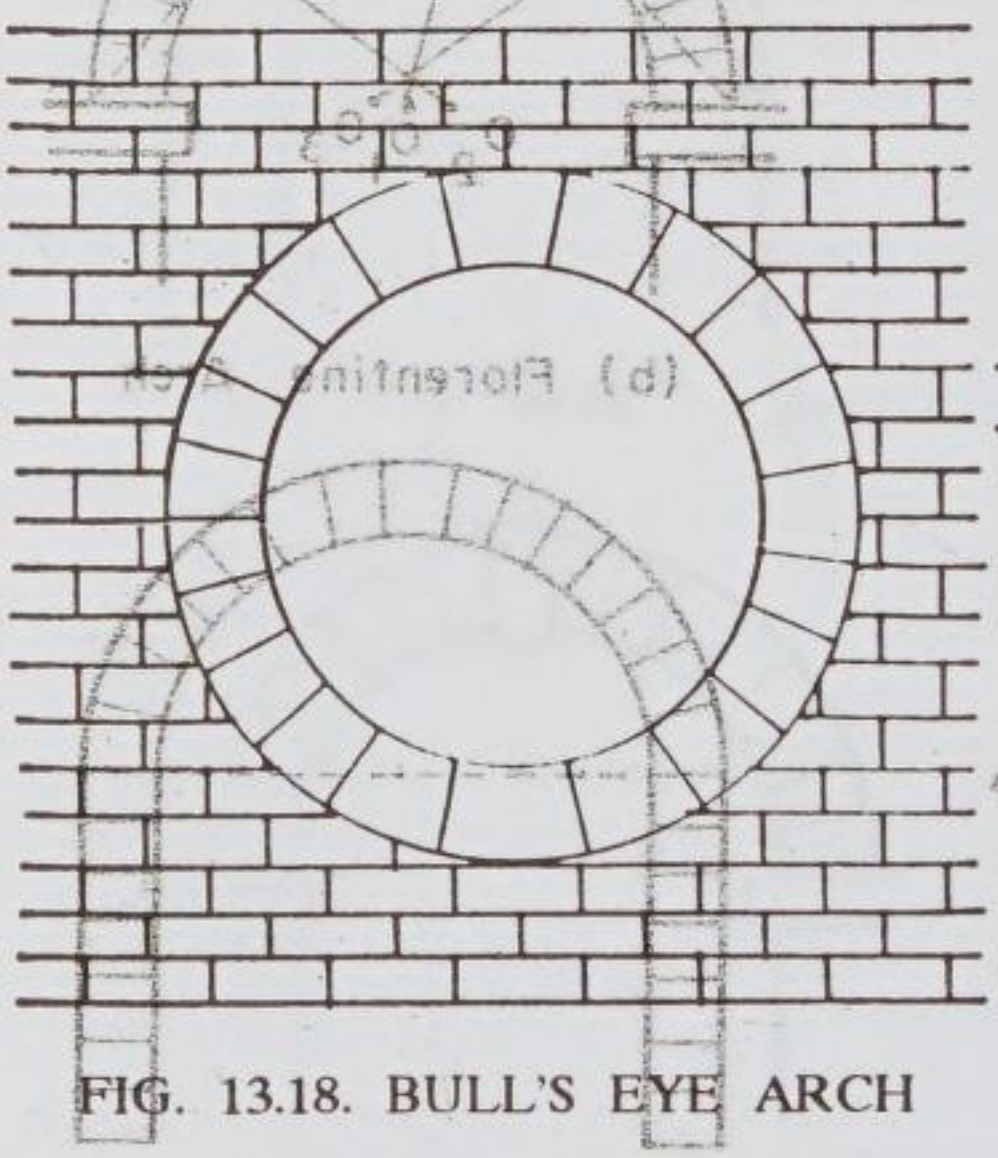


FIG. 13.18. BULL'S EYE ARCH

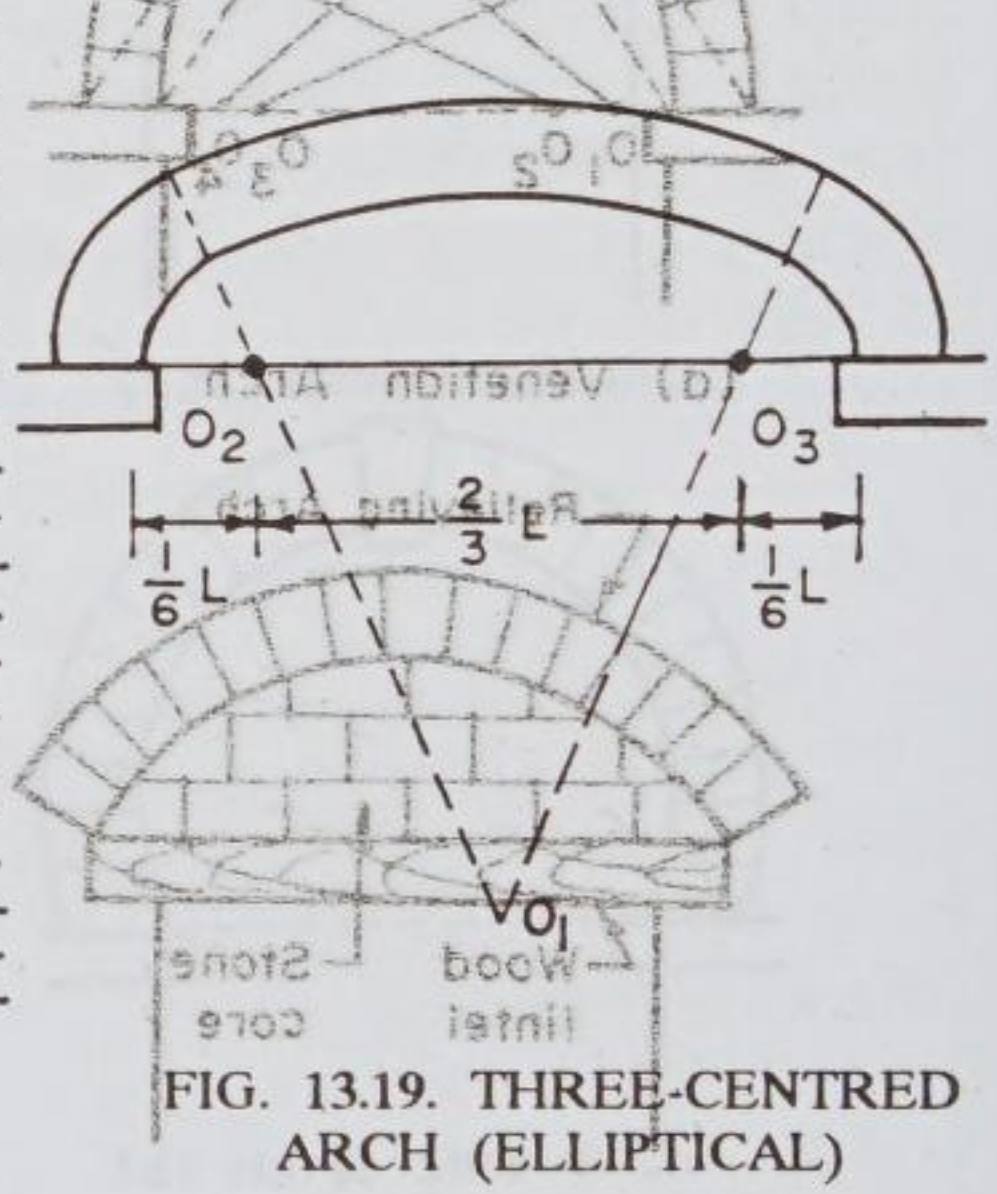


FIG. 13.19. THREE-CENTRED ARCH (ELLIPTICAL)

4. Four-centred arch

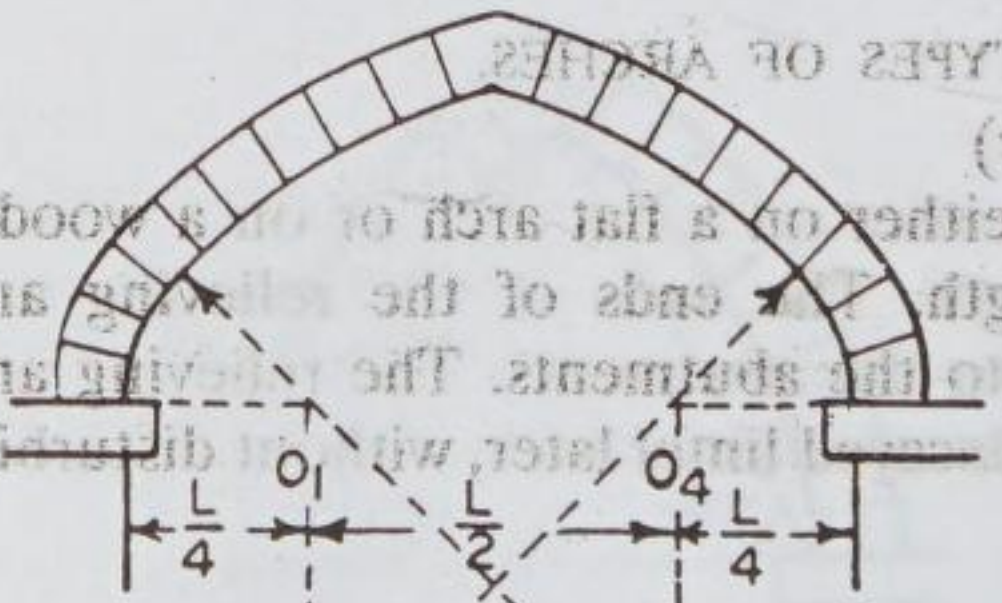


FIG. 13.20. FOUR-CENTRED ARCH (TUDOR ARCH)

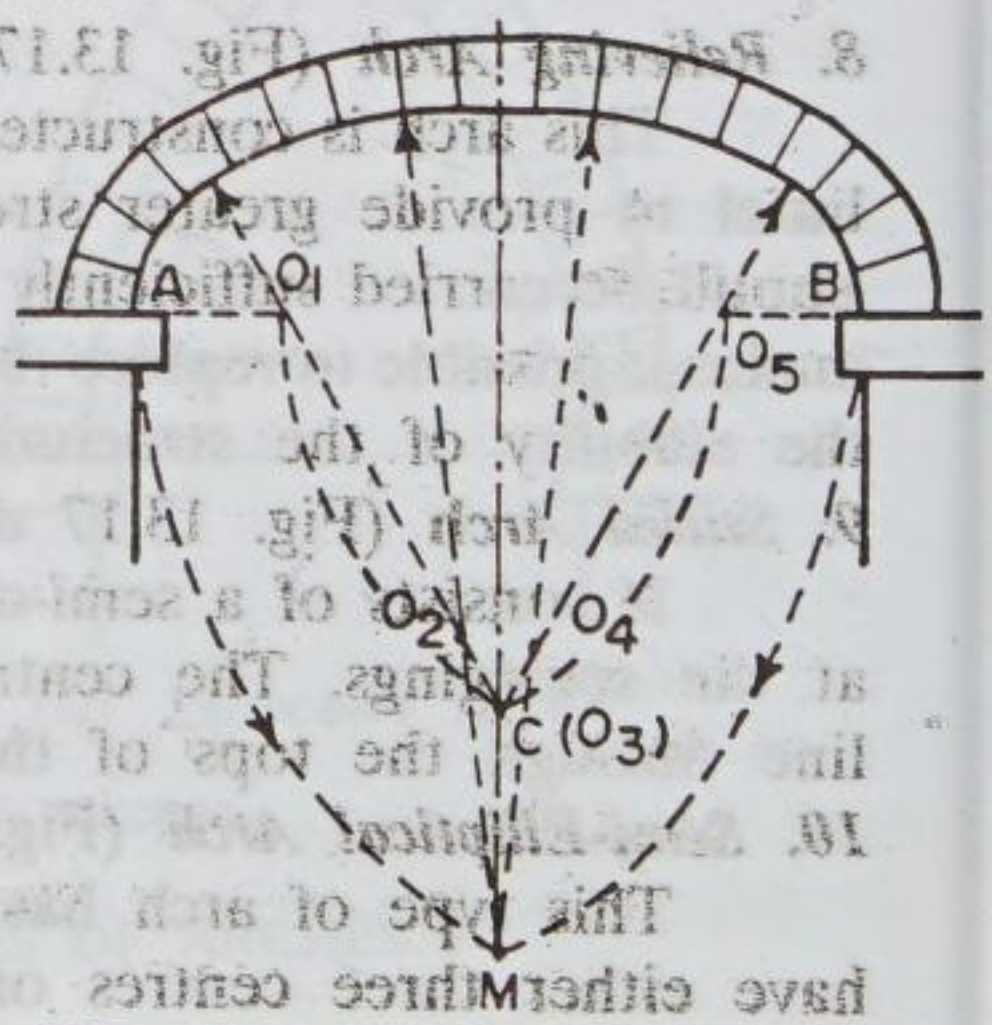


FIG. 13.21. FIVE-CENTRED ARCH (SEMIL-ELIPTICAL ARCH)

It has four centres. Venetian arch is a typical example of this type. Another examples are the *Tudor arch* (Fig. 13.20).

5. ***Five- centred arch.*** This type of arch, having five centres, gives a good semi- elliptical shape.

(*c*) Classification based on material and workmanship

On the basis of material of construction and workmanship, arches may be classified as follows :

1. *Stone arches*
   - (*i*) Rubble arch
   - (*ii*) Ashlar arch.
2. *Brick arches*
   - (*i*) Rough arch
   - (*ii*) Axed or rough-cut arch
   - (*iii*) Gauged arch
   - (*iv*) Purpose made brick arch.
3. *Concrete arches*
   - (*i*) Concrete block-units arch
   - (*ii*) Monolithic arch.

These types are being described in the subsequent articles.

13.12. STONE ARCHES

Depending upon workmanship, stone arches are of two types: (*i*) Rubble arches, and (*ii*) Ashlar arches.

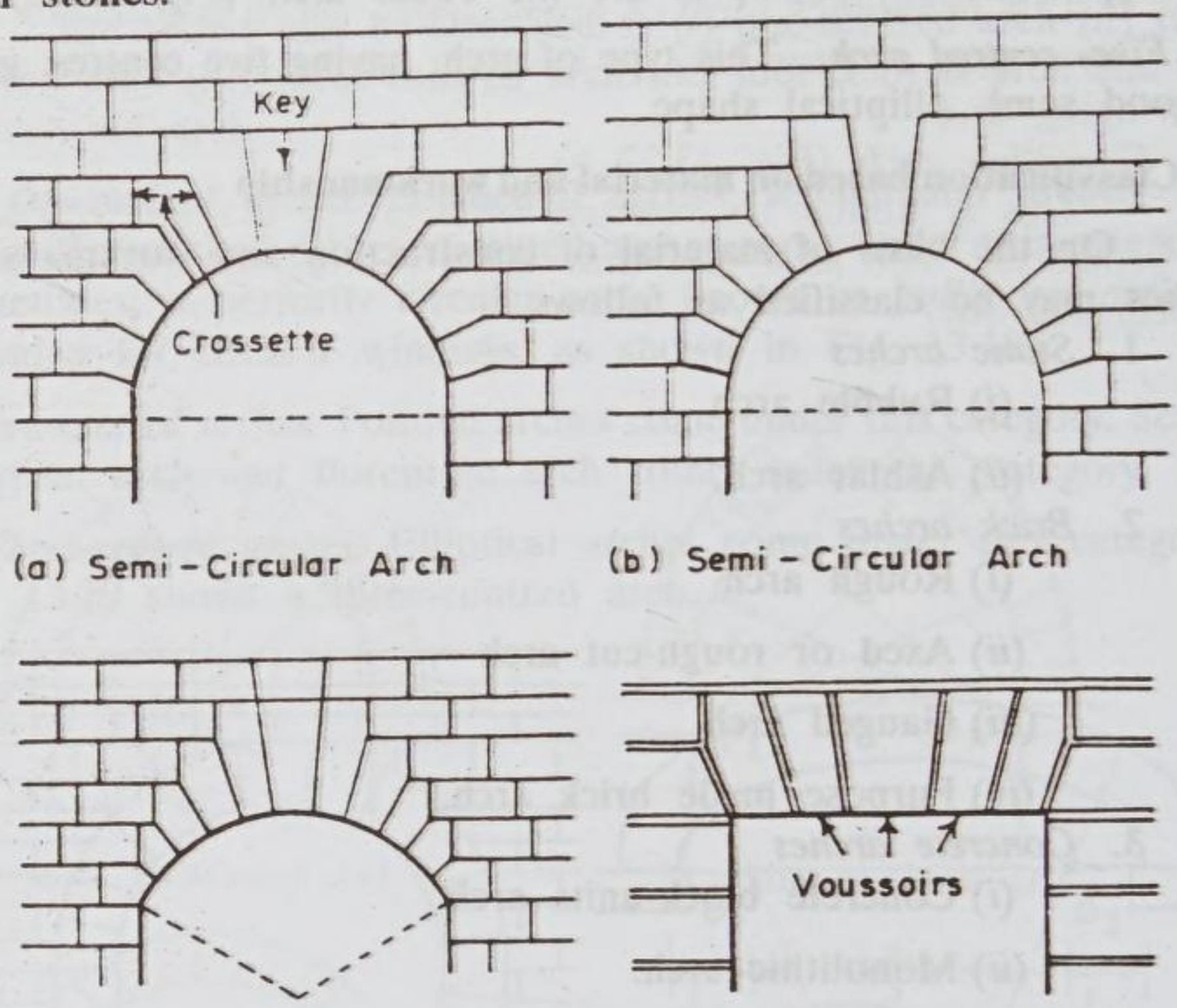
1. Rubble Arches

Rubble masonry arch is comparatively weak and is used for comparatively inferior work. These arches are made of rubble stones which are hammer dressed, roughly to shape and size of voussoirs of the arch and fixed in cement mortar. Rubble arches are used upto spans of 1 m. They are also used as relieving arches, over wooden lintels. Upto a depth of 37.5 cm, these arches are constructed in one ring. For greater depths (thickness), rubble stones are laid in two rings in alternate course of headers and stretchers.

2. Ashlar Arches

In this type, the stones are cut to proper shape of voussoirs, and are fully dressed, set in lime or cement joints with proper bed joints. Upto depth of 60 cm, the voussoirs are made of full thickness of the arch. For determining the wedged shapes of voussoirs, it is preferable to set out the arch on a level platform, marking on it the keystone and voussoirs along with radial mortar joints. Fig. 13.22

shows some details of semicircular, segmental and flat arches of ashlar stones.



(a) Semi-Circular Arch (b) Semi-Circular Arch

(c) Segmental Arch (d) Flat Arch

FIG. 13.22. ASHLAR STONE ARCHES.

Ashlar stone can also be used to make flat arches, in which the joints are either joggled or rebated, as shown in Fig. 13.23. Fig. 13.22 (*d*) shows the alternate arrangement of voussoirs.

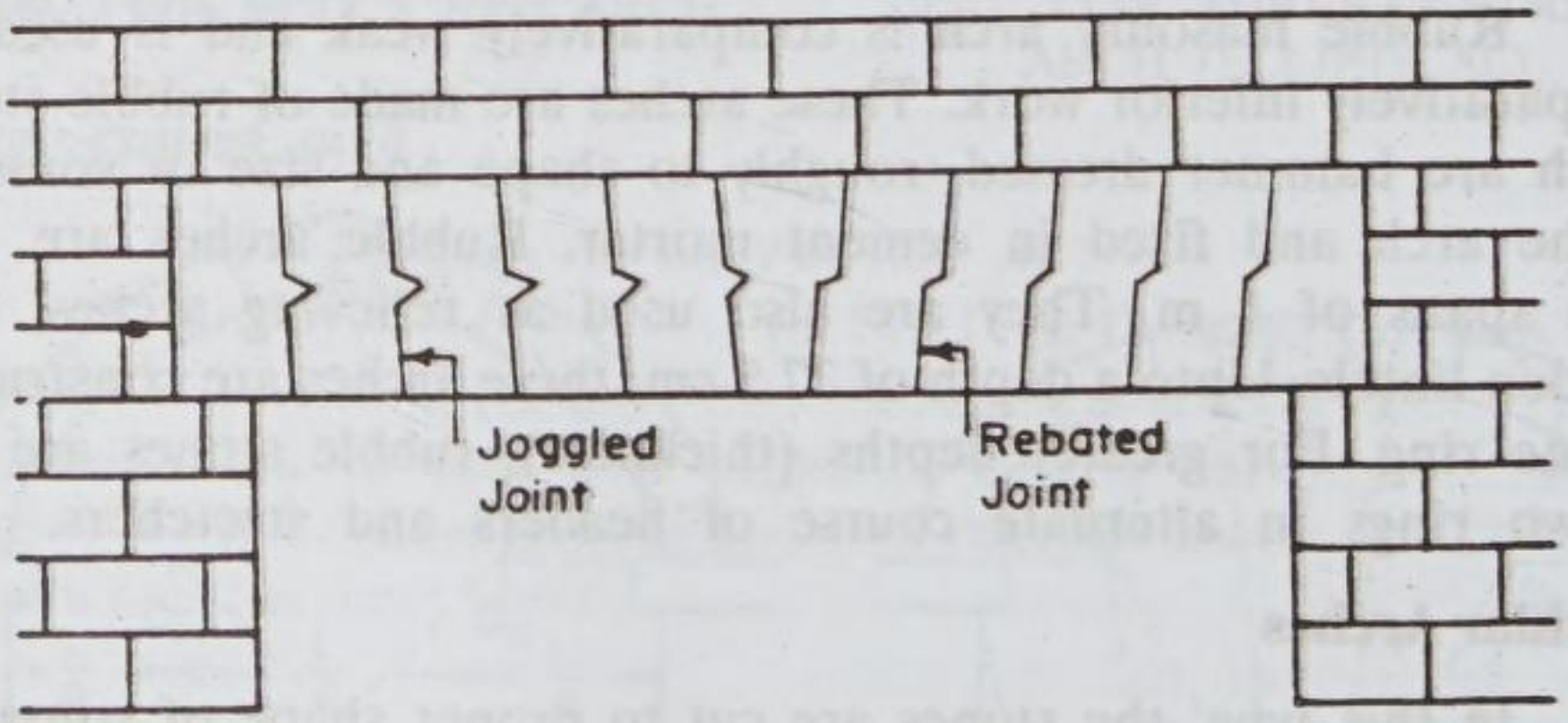


FIG. 13.23. JOGGLED AND REBATED JOINTS IN FLAT ARCH OF ASHLAR STONES

13.13. BRICK ARCHES

Brick arches may be classified as rough brick arches, axed or rough cut brick arches, gauged brick arches and purpose made

brick arches, depending upon the nature of workmanship and quality of bricks used.

1. Rough Brick Arches (Fig. 13.24).

This type of arch is constructed with ordinary bricks, without cutting these to the shape of voussoirs. In order to provide the arch curve, the joints are made wedge-shaped, with greater thickness at the extrados and smaller thickness in intrados. Due to this, the appearance of the arch is spoiled. Therefore, this type of arch is not used for exposed brick work.

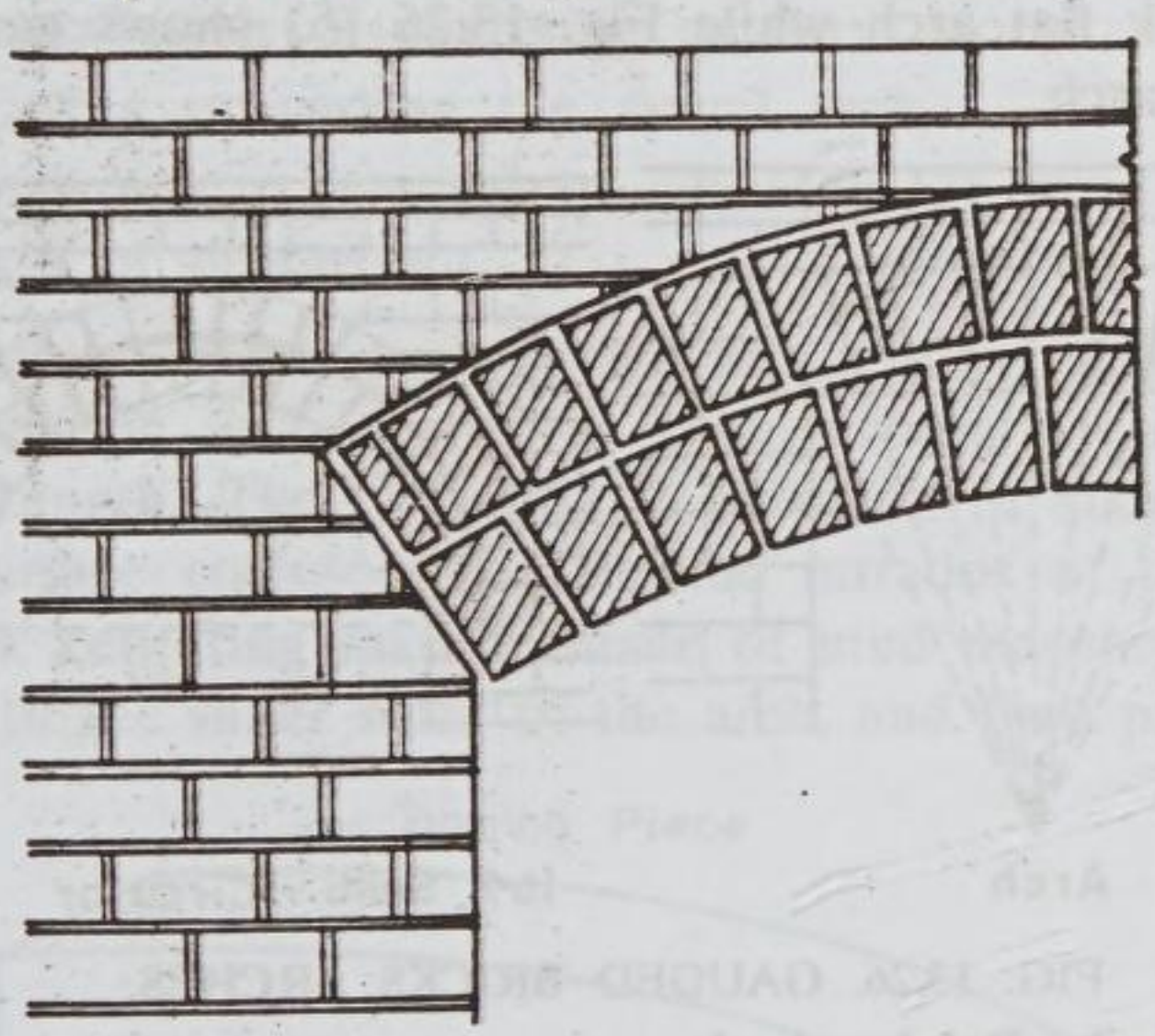


FIG. 13.24. SEGMENTAL ROUGH BRICK ARCH

2. Axed Brick Arches (Fig. 13.25)

In this arch, the bricks are cut wedge-shaped with the help of brick axe. Due to this the joints are of uniform thickness along

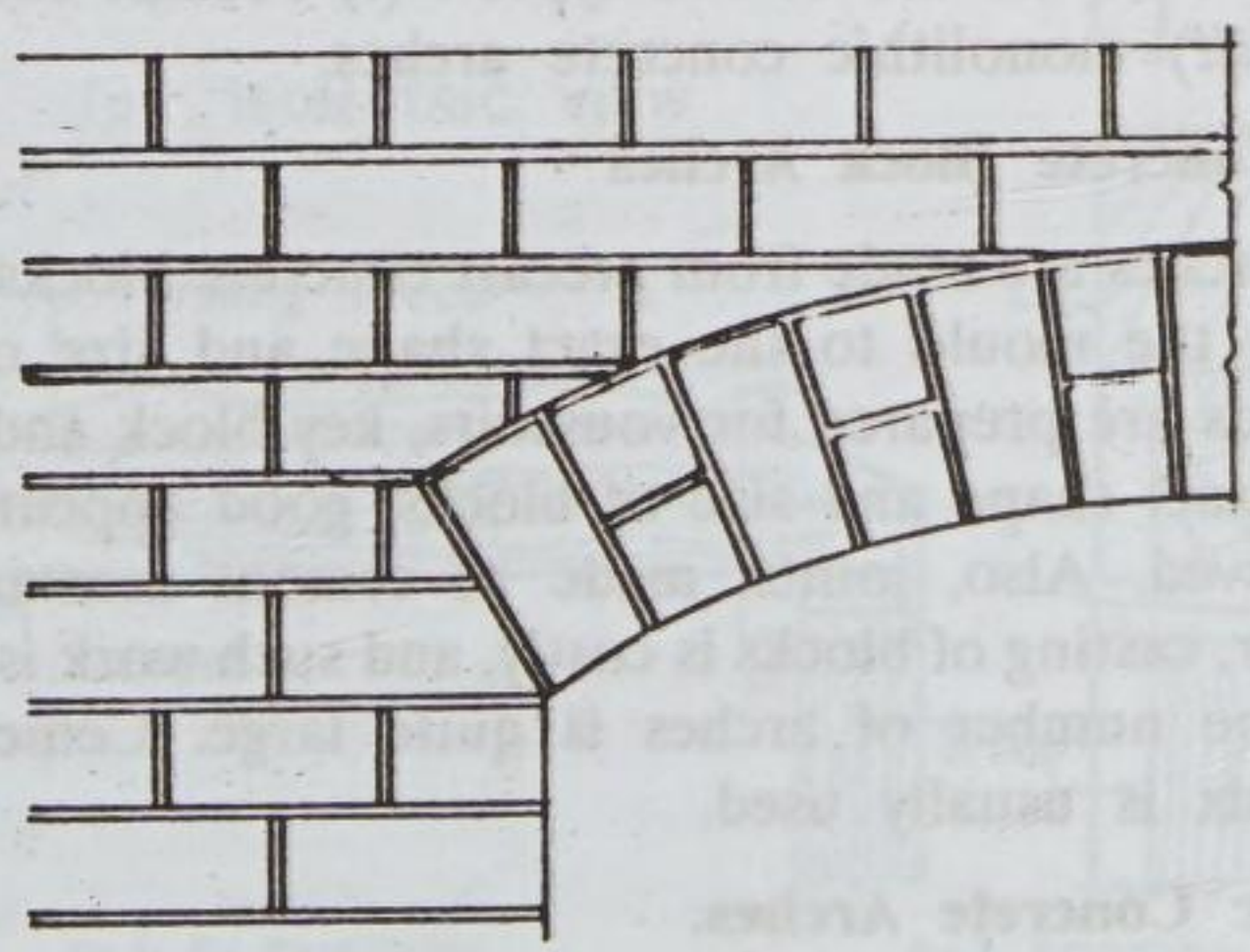


FIG. 13.25. AXED BRICK ARCH.

the radial line. However, the appearance of the arch is not very pleasant because the bricks cut to wedge-shapes are not finely dressed.

3. Gauged Brick Arch

This type of arch is constructed of bricks which are prepared to exact size and shape of voussoir by cutting it by means of wire saw. The surfaces of the bricks are fine dressed with the help of a file. For this, only soft brick (called *rubber bricks*) are used. The joints formed in gauged brick arch are fine, thin (1 to 1.5 mm) and truly radial. Lime putty is used for jointing. Fig. 13.26 (*a*) shows a gauged brick flat arch while Fig. 13.26 (*b*) shows gauged bricks semi-circular arch.

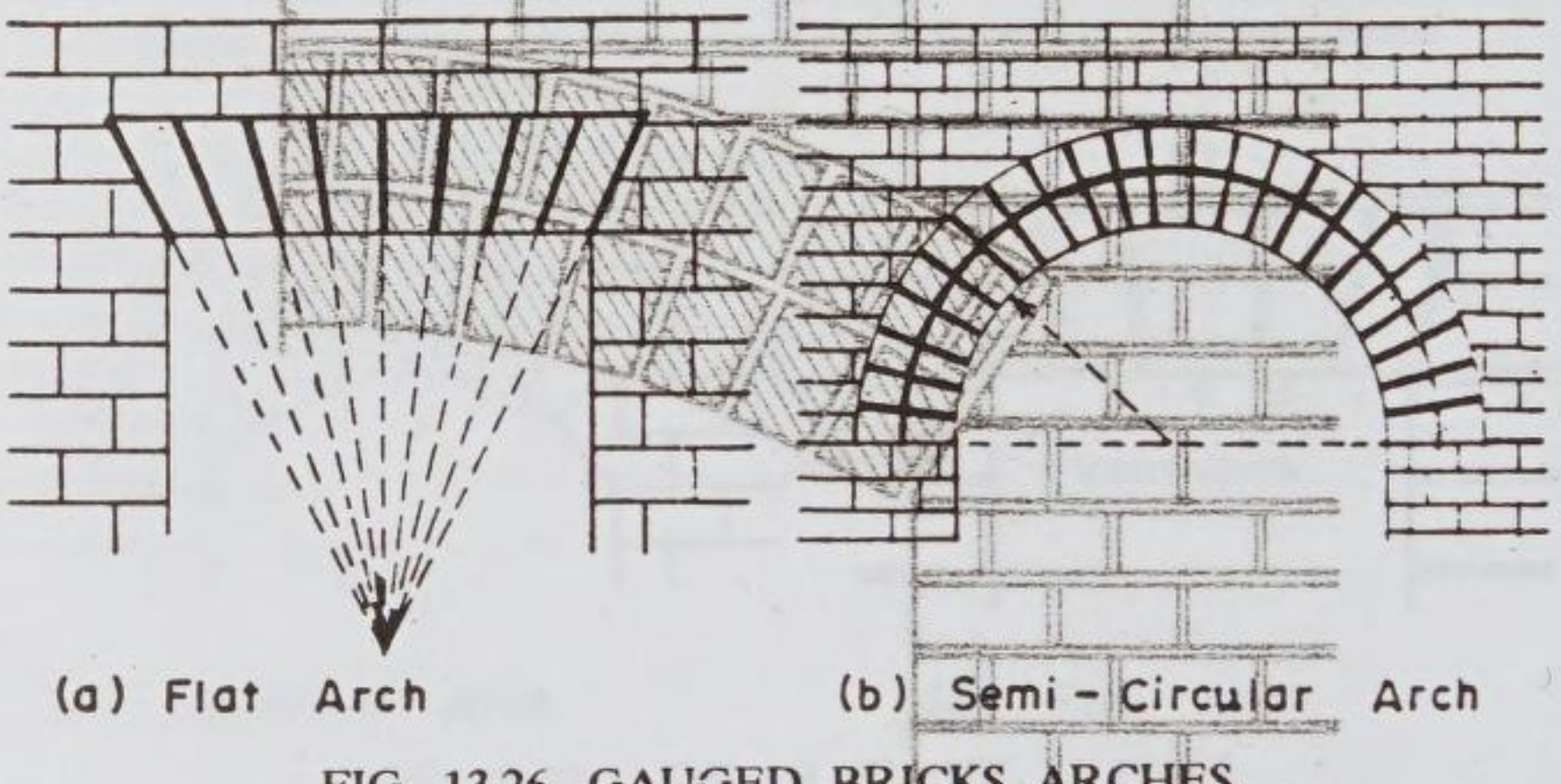


FIG. 13.26. GAUGED BRICKS ARCHES.

4. Purpose made Bricks Arch

In this type of arch, the bricks are manufactured, matching with the exact shape and size of voussoirs, to get a very fine workmanship. Lime putty is used for jointing.

13.14. CONCRETE ARCHES

Concrete arches are of two types : (*i*) Precast concrete block arches, and (*ii*) monolithic concrete arches.

1. Precast Concrete Block Arches

Such arches are made from precast concrete blocks, each block being cast in the mould to the exact shape and size of voussoirs. Special moulds are prepared for voussoirs, key block and skewbacks. Because of exact shape and size of blocks, good appearance of the arch is achieved. Also, joints, made of cement mortar, are quite thin. However, casting of blocks is costly, and such work is economical only when the number of arches is quite large. Cement concrete of 1 : 2 : 4 mix is usually used.

2. Monolithic Concrete Arches.

Monolithic concrete arches are constructed from cast-in-situ

concrete, either plain or reinforced, depending upon the span and magnitude of loading. These arches are quite suitable for larger span. The arch thickness is 15 cm for arches upto 3 m span. Form work is used for casting the arch, and is removed only when the concrete has sufficiently hardened and gained strength. The curing is done for 2 to 4 weeks.

13.15 . CONSTRUCTION OF ARCHES

The construction of arches, of all the types of materials (*i.e.* bricks, stones concrete) is carried out in three steps :

(*i*) Installation of centering or form work

(*ii*) Laying or casting the actual arch

and (*iii*) Striking or removal of centering or form work.

1. Installation of centering

Centering is the temporary structure required to support brick, stone or concrete arch during its construction, till it has gained sufficient strength. The centering is installed in such a way that its upper surface corresponds with the intrados of the arch. For minor works, centering may be made of mud masonry constructed to match with the inner soffit of the arch, and then plastered. This

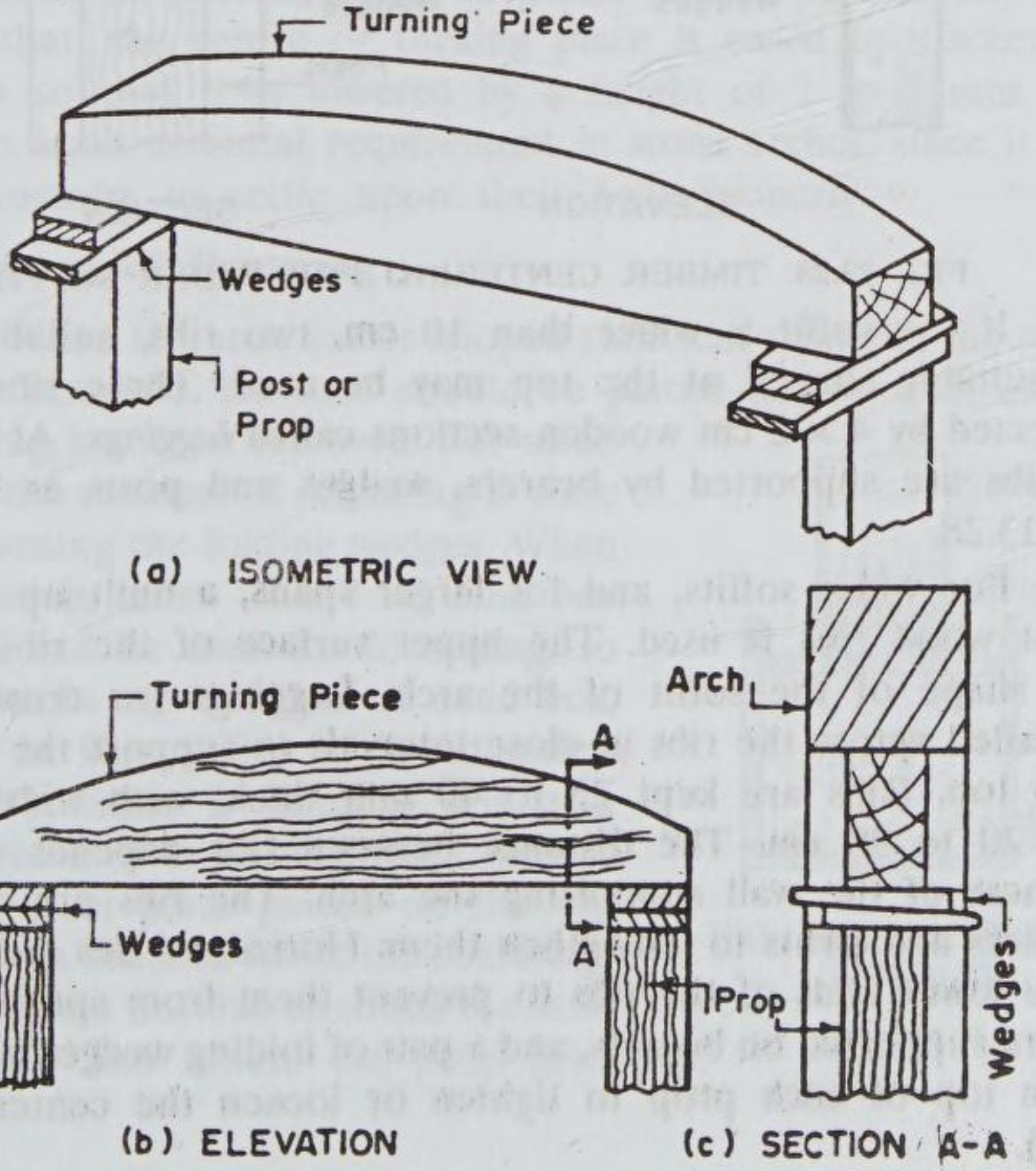


FIG. 13.27. TIMBER CENTERING FOR 10 CM WIDE SOFFITS

masonry is dismantled later when the arch has been constructed and cured.

The usual centering is made of timber or steel. Wooden centering is the simplest and cheapest, used for moderate span. It is easy to construct and easy to dismantle and it can be used several times. Fig. 13.27 shows a thick wooden plank, with horizontal bottom and the upper surface shaped to the underside of the soffit. Such a plank is known as *centre or turning piece.* Its width is normally 10 cm, and is supported on vertical timber posts called props, with wooden wedges to tighten or loosen the centering.

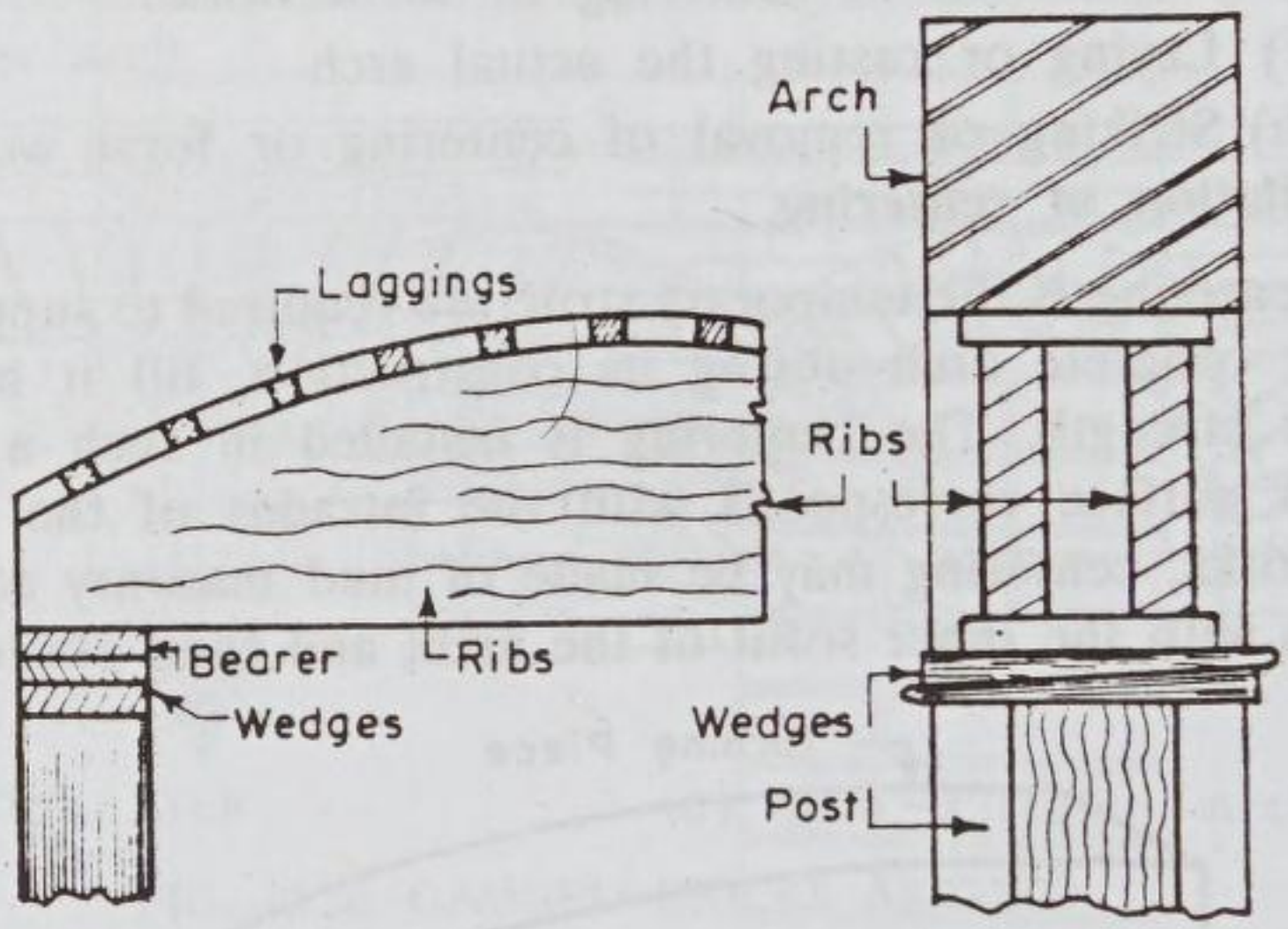


FIG. 13.28. TIMBER CENTERING FOR WIDER SOFFITS

If the soffit is wider than 10 cm, two ribs, suitably spaced and suitably shaped at the top may be used. These ribs may be connected by 4 × 2 cm wooden sections called *laggings*. At the ends, the ribs are supported by bearers, wedges and posts as shown in Fig. 13.28.

For wider soffits, and for larger spans, a built up centering of cut wood ribs is used. The upper surface of the ribs is given the shape of the soffit of the arch. Laggings (or cross-battens) are nailed across the ribs at close intervals to support the voussoirs at its top. Ribs are kept 25 to 40 mm thick, with width varying from 20 to 30 cm. The distance between ribs depends upon the thickness of the wall supporting the arch. The ribs are connected by braces and struts to strengthen them. Horizontal ties are provided at the lower ends of the ribs to prevent them from spreading. The ribs are supported on bearers, and a pair of folding wedges is provided at the top of each prop to tighten or loosen the centering (Fig. 13.29).

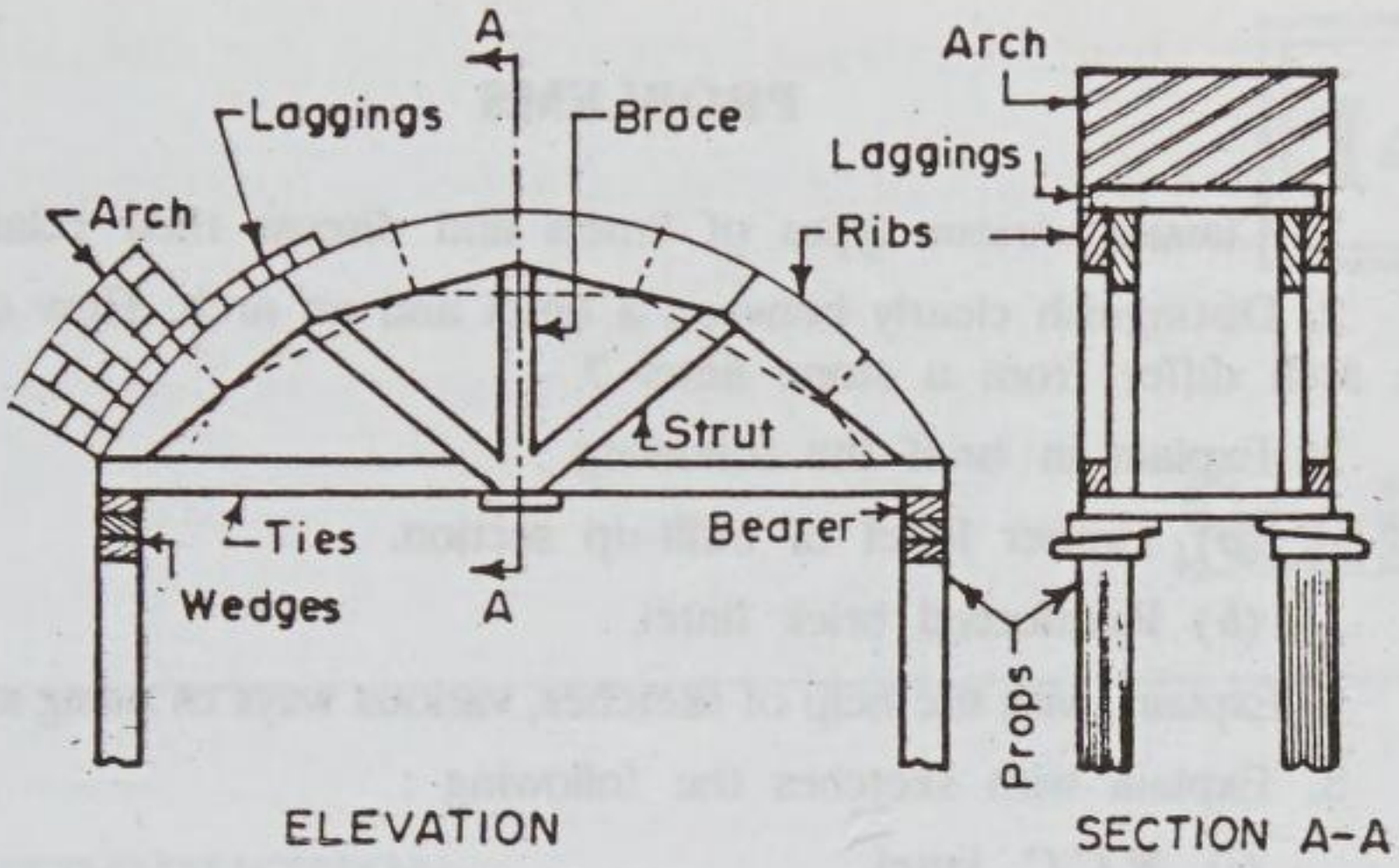


FIG. 13.29. CENTERING FOR WIDE SOFFITS AND BIGGER SPANS.

2. Laying of Arch

After the erection or installation of centering, skewbacks are first prepared. Voussoirs are then arranged in proper and required forms, starting from skewbacks and proceeding towards the crown. Finally, key-stone (or key brick) is inserted so that all the voussoirs are locked in position. The voussoirs should be properly bedded. After that, the centre or turning piece is eased by slackening the wedges so that it is lowered by a height of 2 to 3 mm. Such a process is an essential requirement in stone arches, since it permits the voussoirs to settle upon their beds properly.

3. Removal of Centering

When the arch has developed sufficient strength, the centering can be removed. No load should be placed on the arch unless the centering has been removed. For small spans, the removal of centering is done by loosening the folding wedges. When the span is more than 7 m, *sand box method* can be used for loosening, so that shocks are avoided. A sand box, shown in Fig. 13.30, is placed below the prop. Sand is filled in box with a plugged hole at its bottom. Prop rests on the steel plate placed on the top of sand. In order to lower the centering, plug is taken out due to which the sand flows out and lowers the prop gradually.

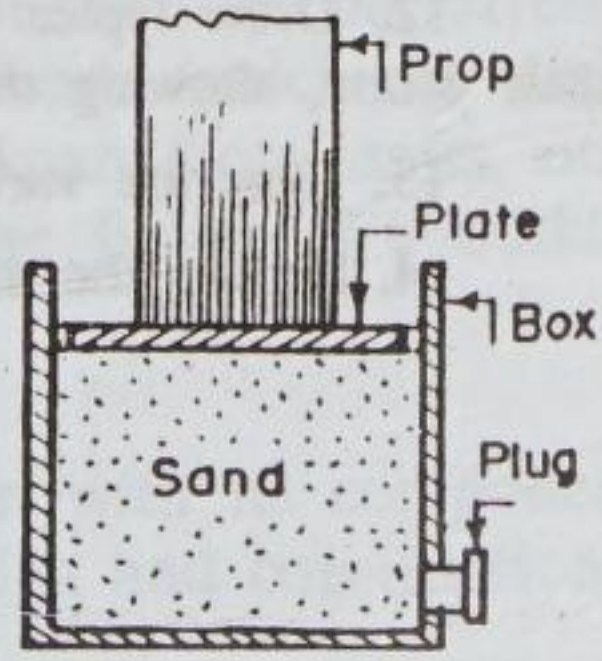


FIG. 13.30 SAND BOX METHOD.

PROBLEMS

1. Classify various types of lintels and discuss their relative use.

2. Distinguish clearly between a lintel and an arch. How does a flat stone arch differ from a stone lintel ?

3. Explain in brief the following :

(*a*) Timber lintel of built-up section.

(*b*) Reinforced brick lintel.

4. Explain, with the help of sketches, various ways of using steel lintels.

5. Explain with sketches the following :

(*i*) R.C.C. lintel.

(*ii*) R.C.C. boot lintel.

6. Discuss various cases of loadings transmitted to a lintel from the wall supported by it.

7. Draw a neat sketch of an arch and show on it various technical terms used in its construction.

8. Discuss various modes of failures of an arch. What are the remedies.

9. Enumerate, with the help of sketches, various types of arches based on its shape.

10. Write short notes on the following :

1. Flat arch.
2. Relieving arch.
3. Bull's eye arch.
4. Elliptical arch.

11. Explain with the help of sketches various types of pointed arches.

12. Draw typical sketches of semi-circular and segmental arches of ashlar stone, showing details of joints between voussoirs.

13. Describe various types of brick arches.

14. Explain the method of erection of centering for arch construction.

14

Stairs

14.1. INTRODUCTION

A *stair* is a set of steps leading from one floor to the other. It is provided to afford the means of ascent and descent between various floors of a building. The room or enclosure of the building, in which the stair is located is known as *stair-case.* The opening or space occupied by the stair is known as a *stairway.* It should be suitably located in a building. In a domestic building the stairs should be centrally located to provide easy access to all the rooms. In public buildings, stairs should be located near the entrance. In big buildings, there can be more than one stairs. Stairs may be constructed of timber, bricks, stone, steel or reinforced cement concrete. However, fire protection of staircases is extremely important. Staircases provide access and communication between floors in multi-storey buildings, and are a path by which fire can spread from one floor to another. Staircase, therefore, must be enclosed by fire resisting walls, floors, ceilings and doors. It is desirable that the linings to the walls and ceiling are non-combustible and of low flame spread. Another important aspect in the design of stairs is the strength aspect. It must be designed to carry certain loads, which are similar to those used for the design of the floor. Apart from stairs, other means of vertical transportation between the floors of a building are : lifts, ramps, ladders and escalators.

14.2. TECHNICAL TERMS

Fig. 14.1 shows the section of a stair, with its components. The technical terms associated with the design and construction of stairs are defined below :

1. ***Step*** : It is a portion of stair which permits ascent or descent. It is comprised of a tread and a riser. A stair is composed of a set of steps.

2. ***Tread*** : It is the upper horizontal portion of a step upon which the foot is placed while ascending or descending.

3. *Riser* : It is the vertical portion of a step providing a support to the tread.

4. *Flight* : This is defined as an unbroken series of steps between landings.

5. *Landing* : It is the level platform at the top or bottom of a flight between the floors. A landing facilitates change of direction and provides an opportunity for taking rest during the use of the stair.

6. *Rise* : It is the vertical distance between two successive tread faces.

7. *Going* : It is the horizontal distance between two successive riser faces.

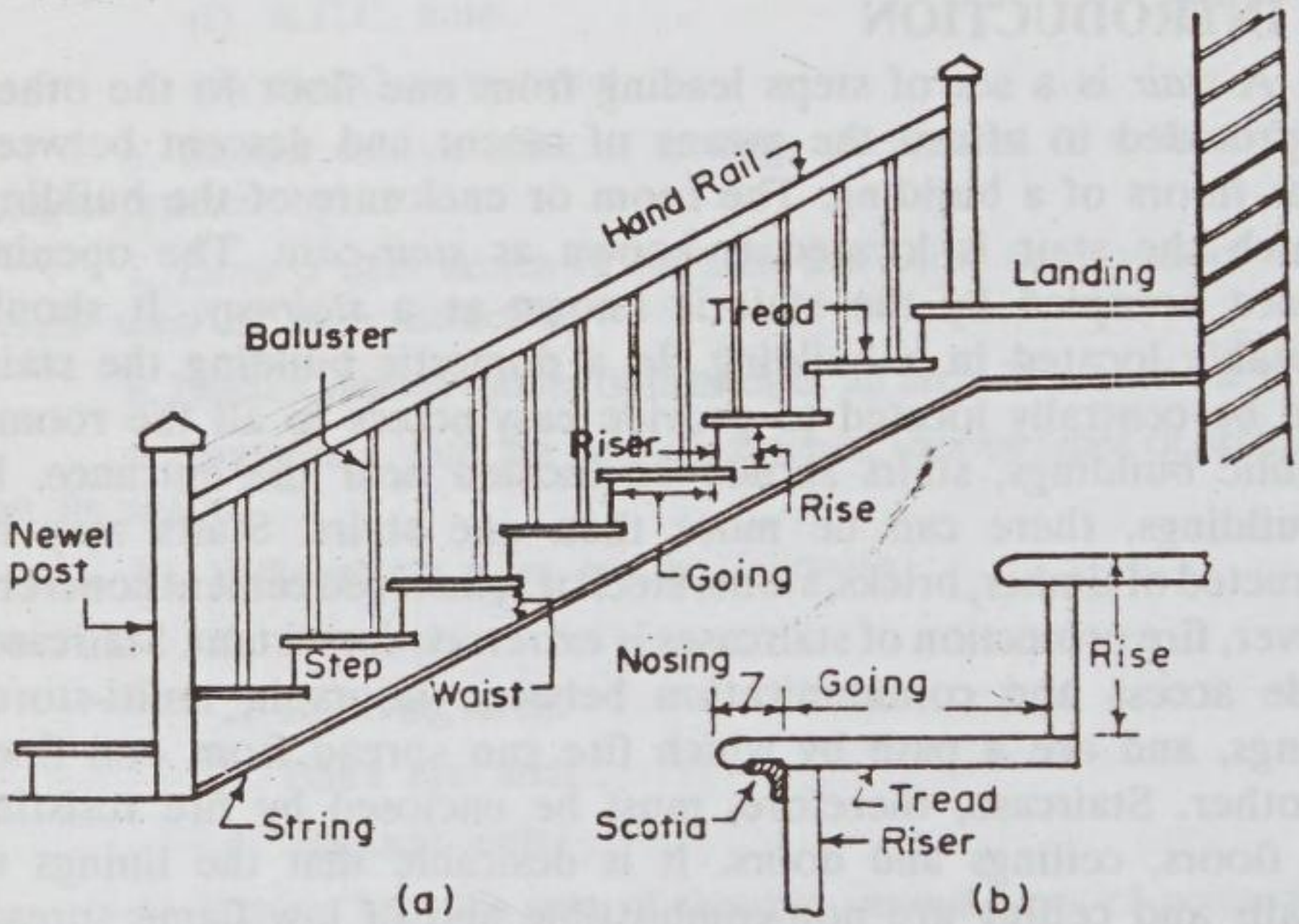


FIG. 14.1 TERMS USED IN STAIRS.

8. *Nosing* : It is the projecting part of the tread beyond the face of the riser. It is usually rounded off from architectural considerations.

9. *Scotia* : It is a moulding provided under the nosing to improve the elevation of the step, and to provide strength to nosing.

10. *Soffit* : It is the underside of a stair.

11. *Line of nosings* : It is an imaginary line parallel to the strings and tangential to the nosings. It is useful in the construction of hand rails, giving the line with which the under-surface of the hand rail should coincide.

12. *Pitch or slope* : It is the angle which the line of nosing of the stair makes with the horizontal.

13. *Strings or stringers* : These are the sloping members which

support the steps in a stair. They run along the slope of the stair.

14. ***Newel post*** **:** Newel post is a vertical member which is placed at the ends of flights to connect the ends of strings and hand rail.

15. ***Baluster*** **:** It is vertical member of wood or metal, supporting the hand rail.

16. ***Balustrade.*** **:** It consists of a row of balusters surmounted by a hand rail, to provide protection for the users of the stair.

17. ***Hand Rail*** **:** It is a rounded or moulded member of wood or metal following generally the contour of the nosing line, and fixed on the top of balusters.

18. ***Head Room*** **:** It is the minimum clear vertical distance between the tread and overhead structure (*i.e.*, ceiling etc.).

19. ***Run*** **:** It is the total length of stairs in a horizontal plane, including landings.

20. ***Header*** **:** It is the horizontal structural member supporting stair stringers or landings.

14.3. REQUIREMENTS OF A GOOD STAIR

Stair is the means of vertical transportation between the floors. It should, therefore, be designed so as to provide easy, quick and safe mode of communication between the floors. Following are the general requirements which a stair should fulfill.

1. Location : (*i*) It should be so located as to provide easy access to the occupants of the building. (*ii*) It should be so located that its well lighted and ventilated directly from the exterior. (*iii*) It should be so located as to have approaches convenient and spacious.

2. Width of stair : It should be wide enough to carry the user without much crowd or inconvenience. Width of stairs depends upto its location in the building and the type of the building itself.

If a domestic building, a 90 cm wide stair is sufficient while in public building, 1.5 to 1.8 m width may be required.

3. Length of flight : From comfort point view, the number of steps are not more than 12 and not less than 3.

4. Pitch of stair : The pitch of the stairs should match with the French theory : 'the labour of moving vertically is about twice that of moving horizontally, if the average human stride is taken as 23 inches. If the rise and going are measured in inch units, the best pitch of the stairs is that inclination which by twicing the rise and adding the going equals 23. When measured in cm units, a comfortable slope is achieved when twice rise plus going is equal to 60 approximately. Pitch should however, be limited to 30° to 45°.

5. Head Room : The clear distance between the tread and

soffit of the flight immediately above it should not be less than 2.1 to 2.3 m, so that even a tall person can use the stair with some luggage on its head.

6. Balustrad : Open well stair should always be provided with balustrade, to provide safety to the users. Wide stair should have hand rail to both the sides.

7. Step dimensions : The rise and going should be of such dimensions as to provide comfort to the users. Their proportion should also be such as to provide desirable pitch of the stair. The going should not be less than 25 cm, though 30 cm going is quite comfortable. The rise should be between 10 cm (for hospitals etc.) to 15 cm. The width of landing should not be less than the width of stair.

8. Materials of construction : The material used for the construction of stair should be such as to provide (*i*) sufficient strength, and (*ii*) fire resistance

14.4. DIMENSIONS OF A STEP

For comfortable ascent and descent, the rise and tread of a step should be well-proportioned. The following thumb rules are followed :

(*i*) (2 × Rise in cm) + (Going in cm) = 60

(*ii*) (Rise in cm) + (Going in cm) = 40 to 45

(*iii*) (Rise in cm) × (Going in cm) = 400 to 450

(*iv*) Adopt Rise = 14 cm and Going = 30 cm as standard ; then for every 20 mm subtracted from going, add 10 mm to the rise.

Thus, other combinations for rise and going would be 15 cm × 28 cm ; 16 cm × 26 cm ; 17 cm × 24 cm.

For residential buildings, the common size of the steps is 16 cm × 26 cm. In hospital etc., the comfortable size of the steps is 10 cm × 30 cm.

Types of steps

Steps in a stair may be of the following of types :

1. Flier
2. Bull nose step
3. Round ended step
4. Splayed step
5. Commode step
6. Dancing step
7. Winder.

A *flier* is an ordinary step of uniform width and rectangular shape in plan, as shown in Fig. 14.2 (*a*). A *bull nose step*, generally provided at the bottom of the flight, projects in front of the newel

post. Its end near the newel forms the quadrant of a circle (Fig. 14.2 *a*). A round ended step is similar to a bull nose step except that it has a semi-circular end which projects out from the stringer. A *splayed step* is also provided at the beginning of the flight, with its end, near the newel post, splayed as shown in Fig. 14.2 (*c*). A *commode step*, shown in Fig. 14.2 (*d*) has curved tread and riser. *Dancing or balancing steps* are the winders which do not radiate from a common centre. *Winders* are tapering steps, such as those which radiate from a point usually situated at the centre of a newel (Fig. 14.2 *f*).

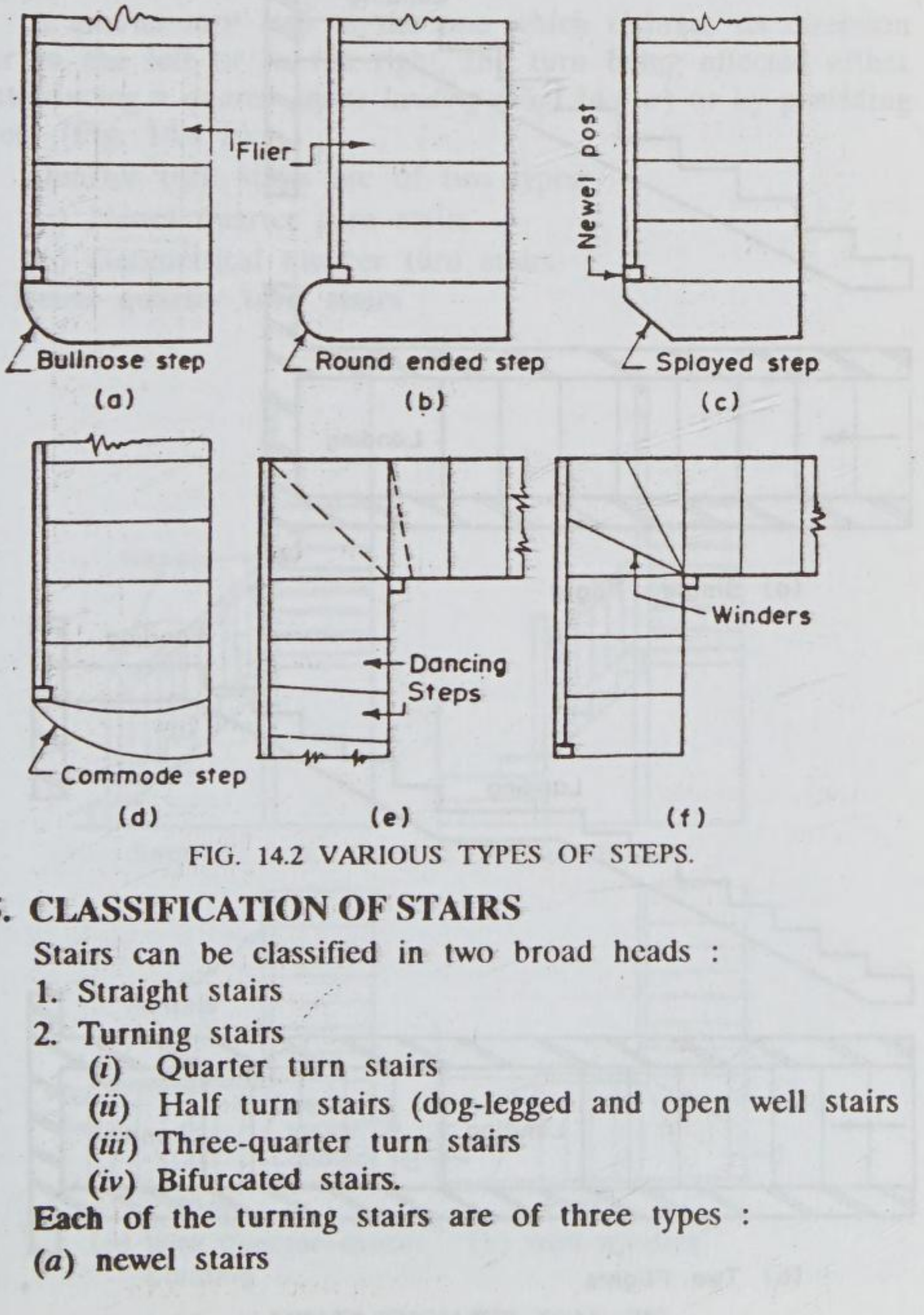


FIG. 14.2 VARIOUS TYPES OF STEPS.

14.5. CLASSIFICATION OF STAIRS

Stairs can be classified in two broad heads :

1. Straight stairs
2. Turning stairs
   - (*i*) Quarter turn stairs
   - (*ii*) Half turn stairs (dog-legged and open well stairs
   - (*iii*) Three-quarter turn stairs
   - (*iv*) Bifurcated stairs.

Each of the turning stairs are of three types :

(*a*) newel stairs

(*b*) well or open-newel stairs, and

(*c*) geometrical stairs

A *newel stair* is the one which has a newel at the foot and head of each flight of the stair, and in which newels are conspicuous features. In *well or open newel stairs,* lateral space is left between the turning flights. Open newel stair present the best appearance and are strong *Geometrical stairs* have the strings and hand rails continuous and are set out in accordance with geometrical principles. They may be *circular, spiral, helical,* or even *elliptical.* A newel may

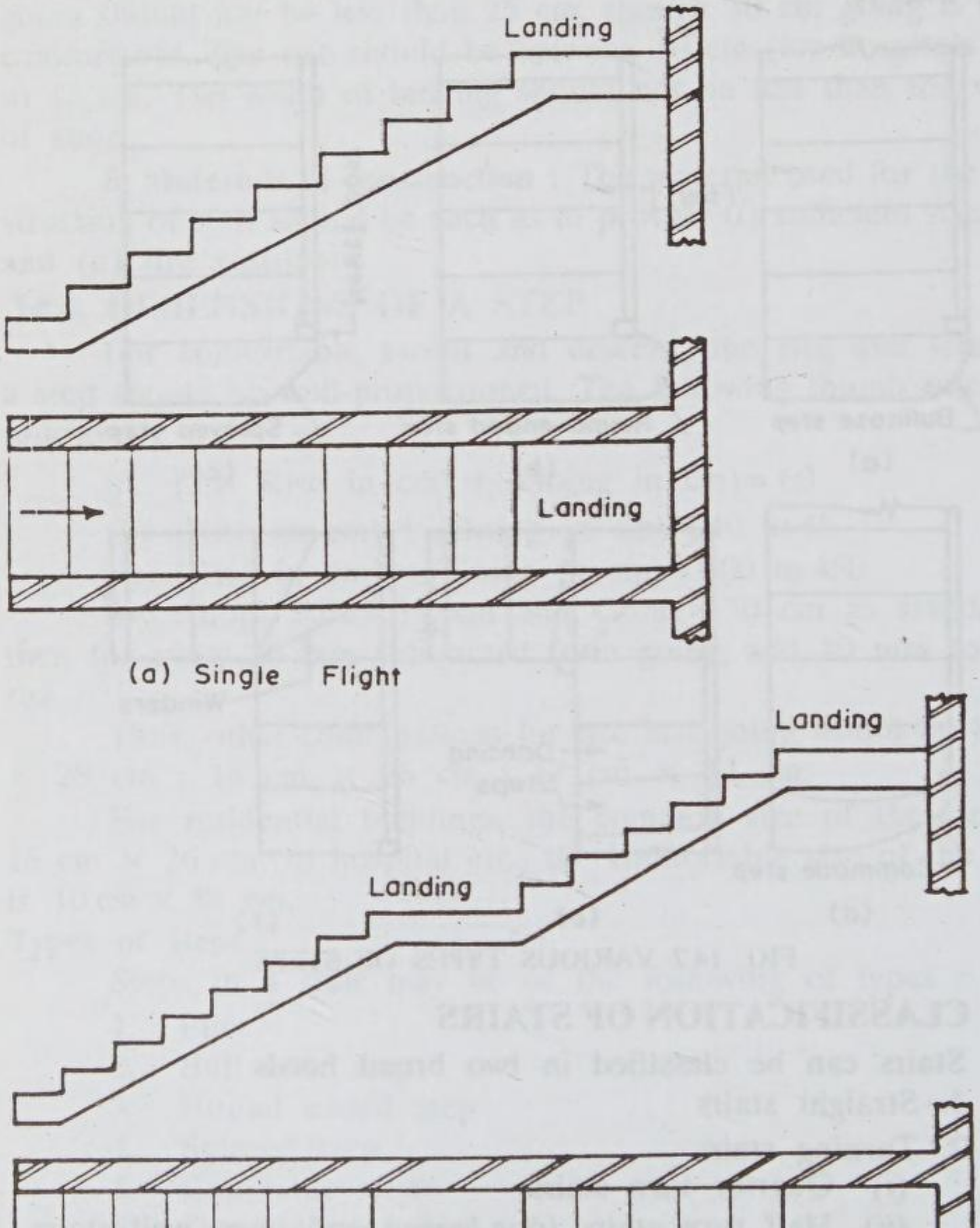


FIG. 14.13. STRAIGHT STAIRS.

be introduced at the bottom and top of such a stair, though it is not an essential part of the construction. Geometrical stairs require care and good deal of skill in their construction. They are not so imposing as the open newel type, and are comparatively weak.

1. STRAIGHT STAIRS

In this type, this stair runs straight between the two floors. It is used for small houses where there are restrictions in available width. The stair may consist of either one single flight or more than one flight (usually two) with a landing, as shown in Fig. 14.3.

2. QUARTER TURN STAIRS

A *quarter turn stair* is the one which changes its direction either to the left or to the right, the turn being affected either by introducing a *quarter space landing* (Fig. 14.4 *a*) or by providing winders (Fig. 14.4 *b*).

Quarter turn stairs are of two types:

(*a*) Newel quarter turn stairs

(*b*) Geometrical quarter turn stairs.

(*a*) Newel quarter turn stairs

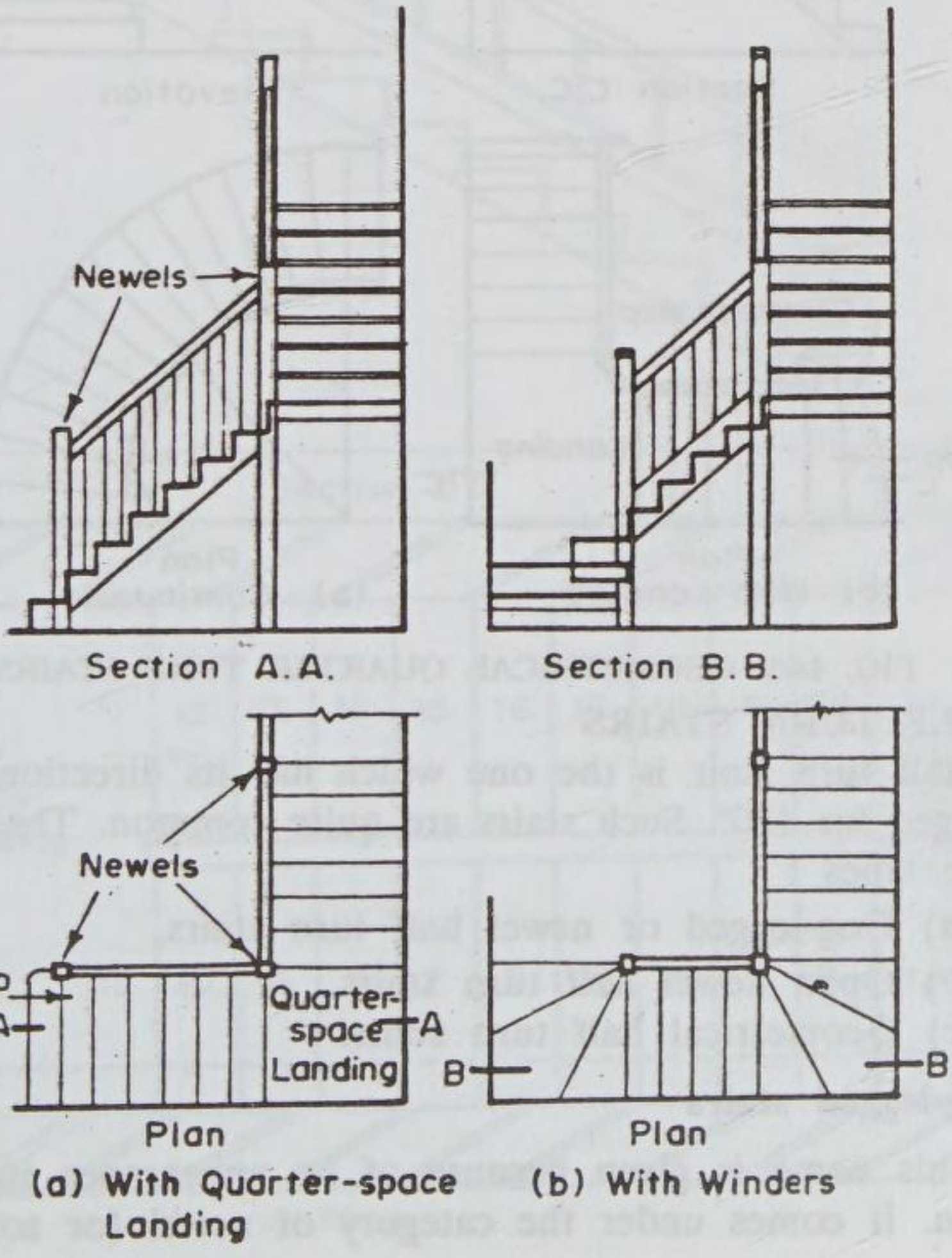


FIG. 14.4. NEWEL QUARTER TURN STAIRS

These stairs have the conspicuous newel posts at the beginning and end of each flight. At the quarter turn, there may either be quarter space landing or there may be winders. Two forms of this type are shown in Fig. 14.4.

(*b*) Geometrical quarter turn stairs

In geometrical stairs, the stringer as well as the hand rail is continuous, with no newel post at the landing. Two forms are shown in Fig. 14.5

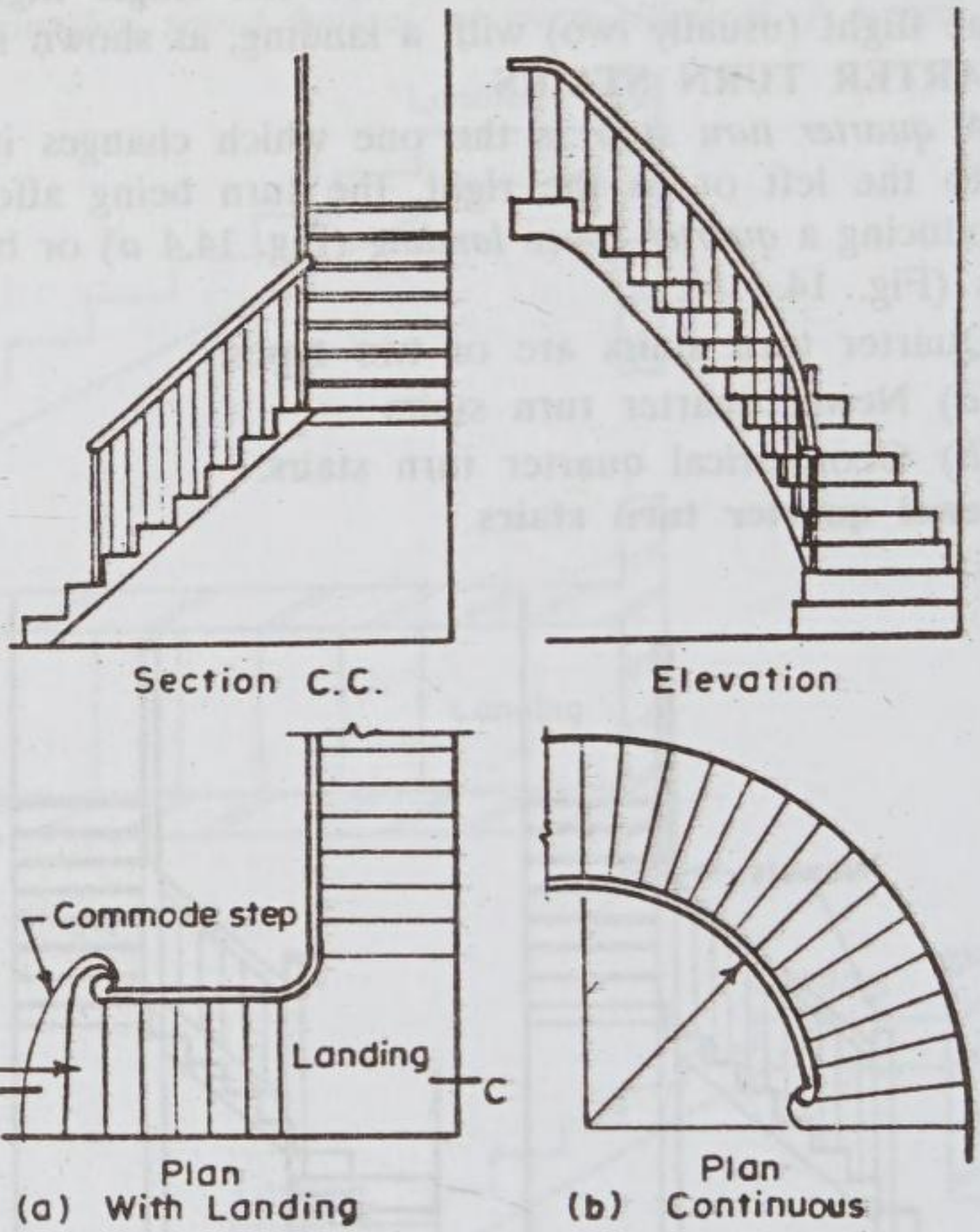


FIG. 14.5. GEOMETRICAL QUARTER TURN STAIRS.

3. HALF TURN STAIRS

Half turn stair is the one which has its direction reversed, or changed for 180°. Such stairs are quite common. These may be of three types :

(*a*) Dog-legged or newel half turn stairs.
(*b*) Open newel half turn stairs.
(*c*) Geometrical half turn stairs.

(*a*) Dog-legged stairs

This name is given because of its appearance in sectional elevation. It comes under the category of newel (or solid newel)

stairs in which newel posts are provided at the beginning and end of each flight. These may be of two forms : (*i*) with half space landing, and (*ii*) with quarter space landing and winders. Generally,

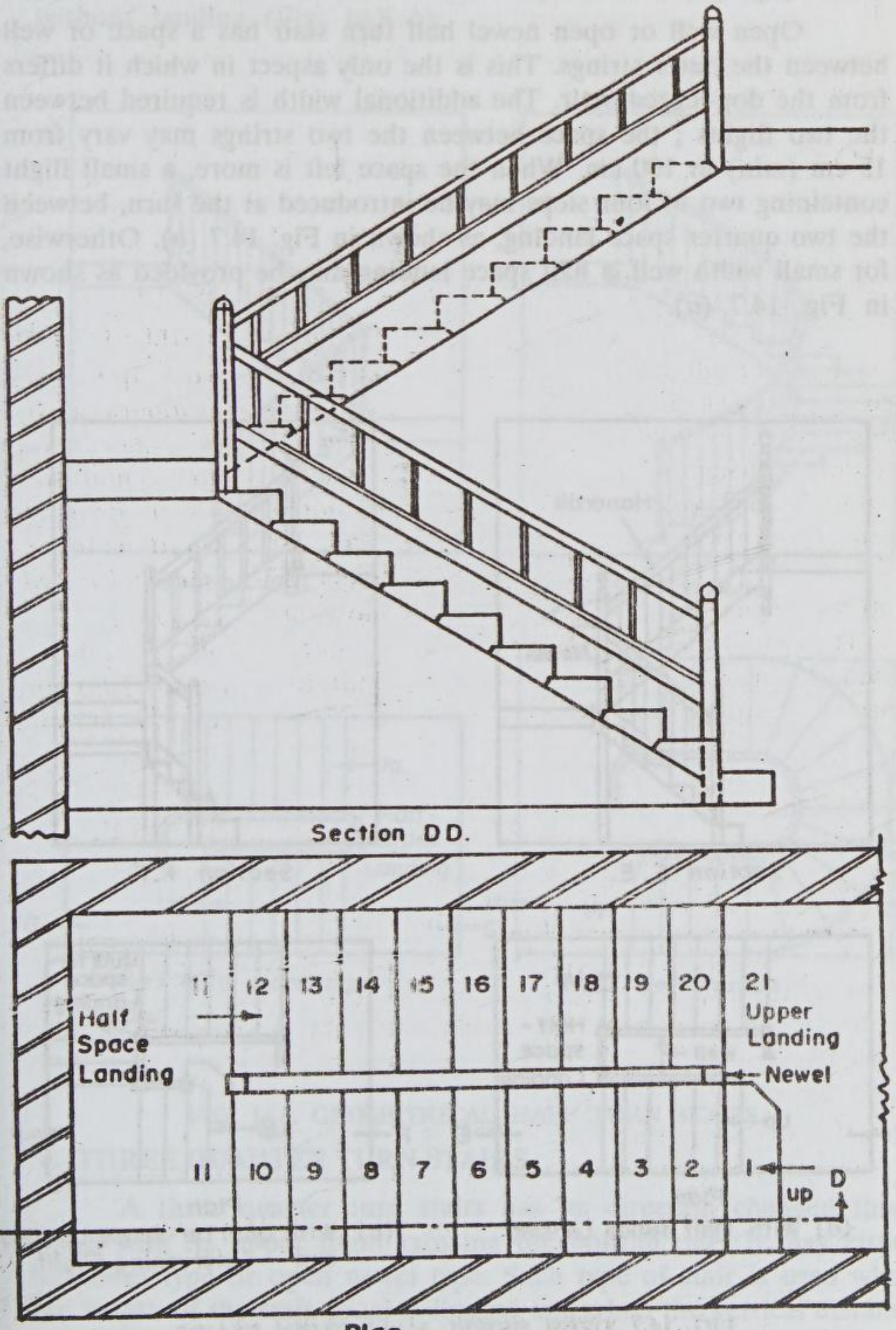


FIG. 14.6. DOG-LEGGED STAIR.

the former type (*i.e.*. without winders) is more common, as shown in Fig. 14.6. There is no space between the outer strings of the two flights.

(b) Open newel half turn stair

Open well or open newel half turn stair has a space or well between the outer strings. This is the only aspect in which it differs from the dog-legged stair. The additional width is required between the two flights ; the space between the two strings may vary from 15 cm (min) to 100 cm. When the space left is more, a small flight containing two to four steps may be introduced at the turn, between the two quarter space landing, as shown in Fig. 14.7 (*b*). Otherwise, for small width well, a half space landing may be provided as shown in Fig. 14.7 (*a*).

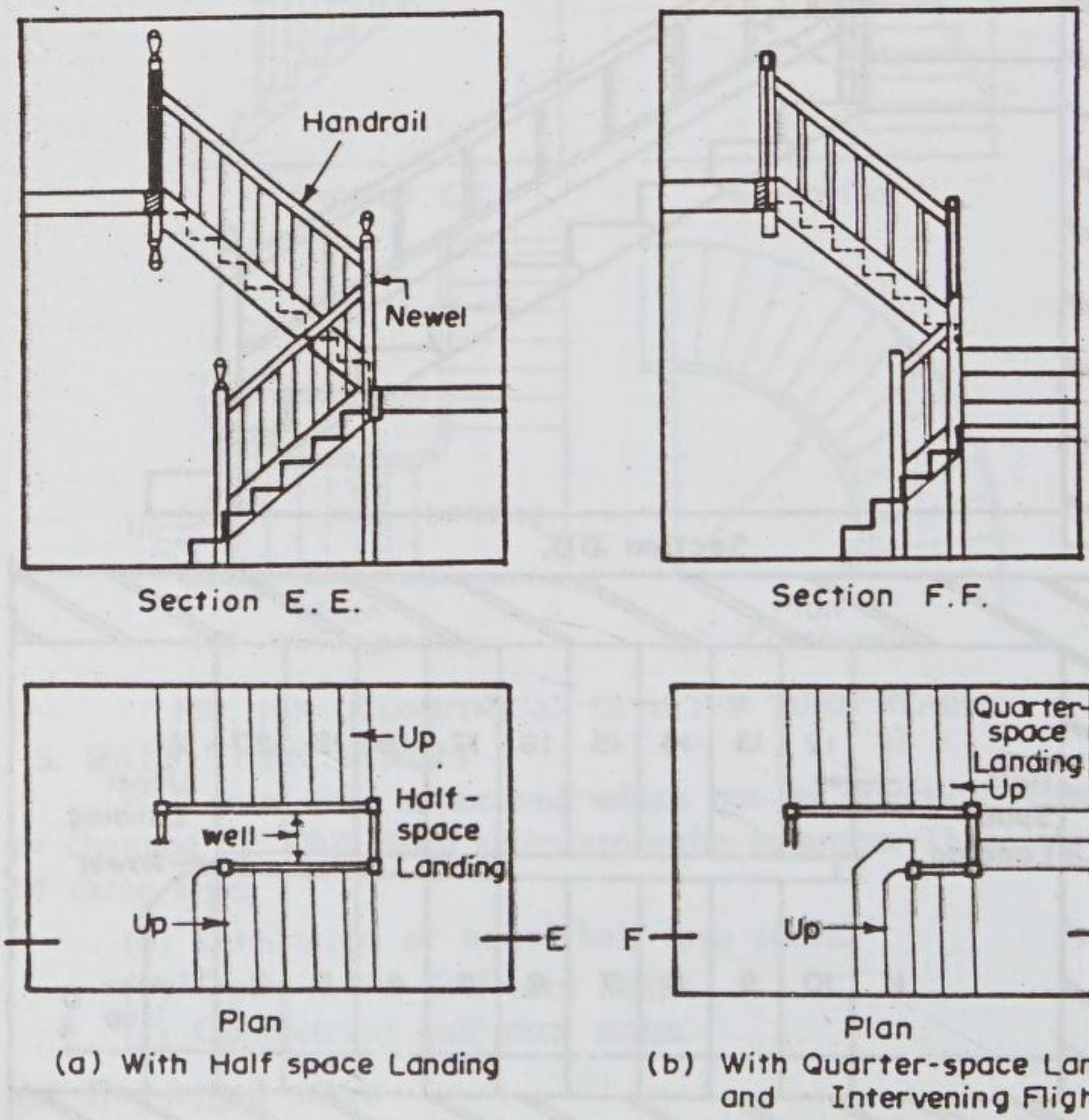


FIG. 14.7. OPEN NEWEL HALF TURN STAIRS.

(*c*) Geometrical half turn stairs

The essential features of such stairs are that the stringers and the hand rails are continuous, without any intervening newel post. These may be either with half-space landing (Fig. 14.8 *a*) or without landing (Fig. 14.8 *b*).

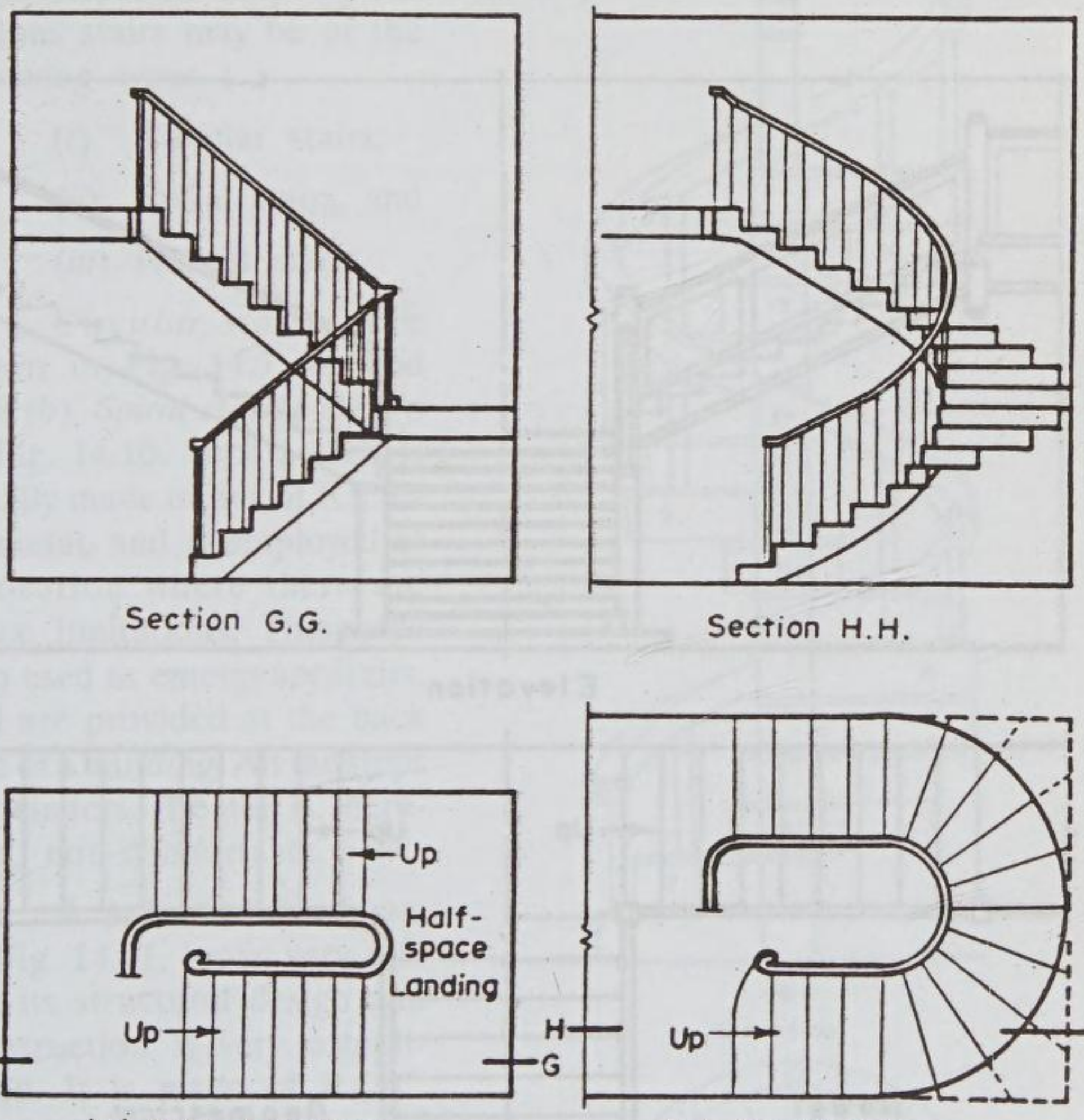


FIG. 14.8. GEOMETRICAL HALF TURN STAITS.

4. THREE QUARTER TURN STAIRS

A three quarter turn stairs has its direction changed three times with its upper flight crossing the bottom one. It may either be newel type or open newel type. Such type of stair is used when the length of the stair room is limited and when the vertical distance between the two floors is quite large.

5. BIFURCATED STAIRS

This type of stair is commonly used in public building at their entrance hall. The stair has a wider flight at the bottom, which bifurcates into two narrower flights, one turning to the left and the other to the right, at the landing. It may be either of newel type with a newel post as shown in Fig. 14.9 (left side) or of geometrical

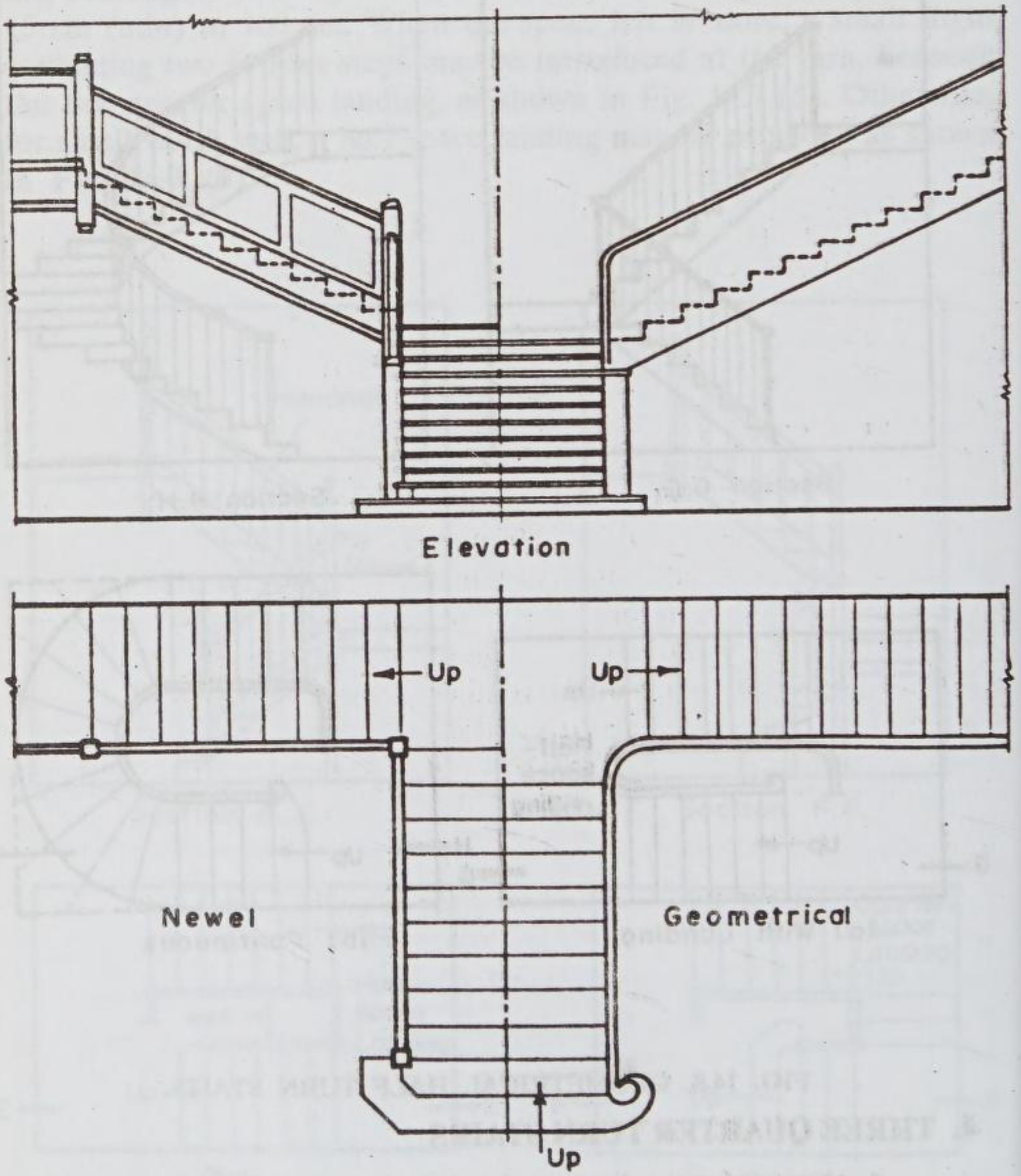


FIG. 14.9. BIFURCATED STAIR.

type, as shown in the right portion of Fig. 14.9 with continuous stringer and hand rails.

6. CONTINUOUS STAIRS

Continuous stairs are those which do neither have any landing nor any intermediate newel post. They are, therefore, geometrical in shape. Continuous stairs may be of the following types :

(*i*) Circular stairs,

(*ii*) Spiral stairs, and

(*iii*) Helical stairs

Circular stairs are shown in Fig. 14.5 (*b*) and 14.8 (*b*). *Spiral stair* is shown in Fig. 14.10. Such a stair is usually made either of R.C.C. or metal, and is employed at a location where there are space limitations. These are also used as emergency stairs, and are provided at the back side of a building. All the steps are winders. The stair is, therefore, not comfortable.

A *helical stair*, shown in Fig. 14.11, looks very fine but its structural design and construction is very complicated. It is made of R.C.C. in which a large portion of steel is required to resist bending, shear and torsion.

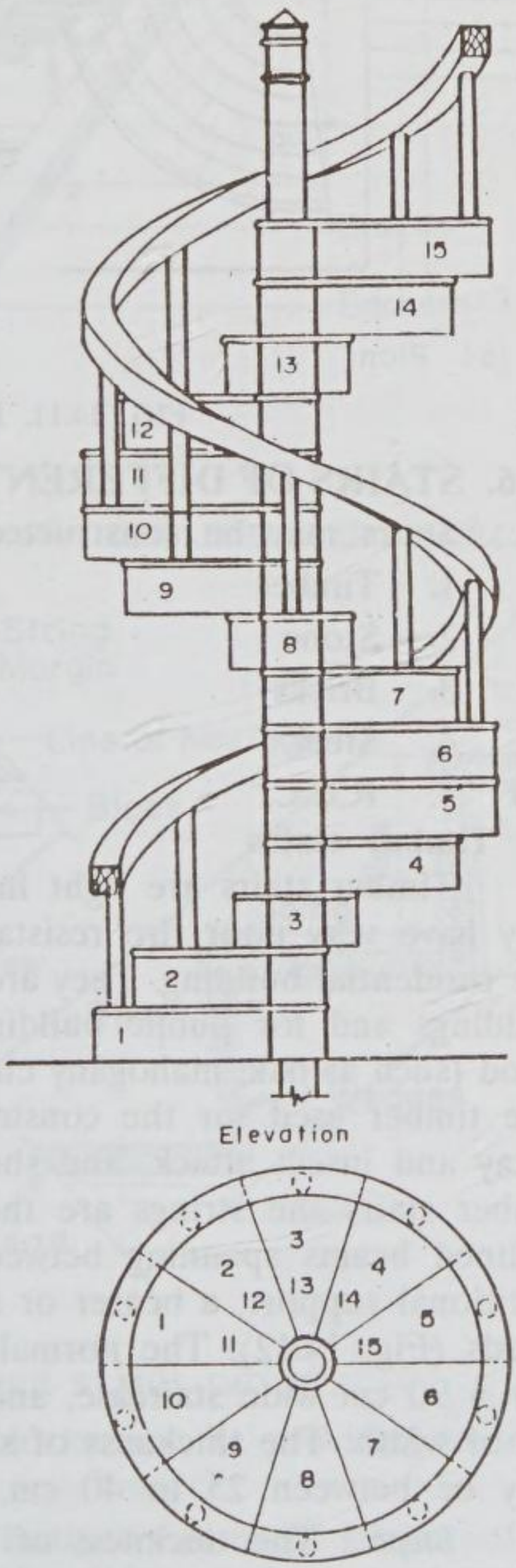


FIG. 14.10 SPIRAL STAIR.

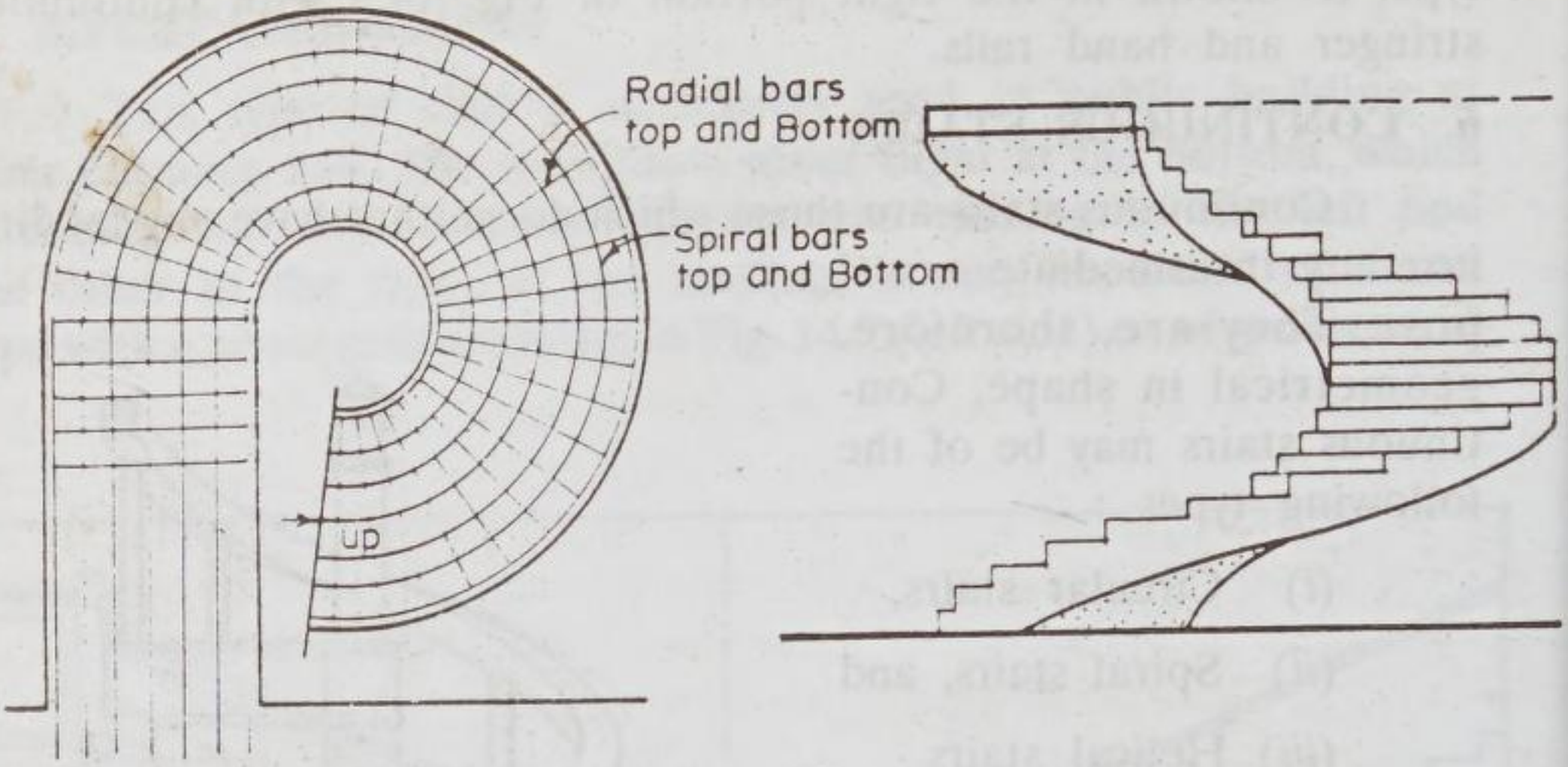


FIG. 14.11. HELICAL STAIR.

14.6. STAIRS OF DIFFERENT MATERIALS

Stairs may be constructed of the following materials :

1. Timber
2. Stone
3. Bricks
4. Steel

and 5. R.C.C.

1. Timber stairs

Timber stairs are light in weight and easy to construct, but they have very poor fire resistance. They are used only for small rise residential building. They are unsuitable for high rise residential buildings and for public buildings. Sometimes, fire resisting hard wood (such as oak, mahogany etc.) of proper thickness may be used. The timber used for the construction should be free from fungal decay and insect attack, and should be well-treated before use. In timber stairs the strings are the support for the stair and act as inclined beams spanning between the floor and the landing. For additional support, a bearer or a carriage may be placed under the treads (Fig. 14.12). The normal practice is to provide one bearer. For a 90 cm wide staircase, and an additional bearer for every 40 cm of width. The thickness of strings may be 3 to 5 cm and depth may be between 25 to 40 cm.

Steps : The thickness of tread of a timber stair should not be less than 32 mm ($1\frac{1}{4}$ inch) and that of riser 25 mm (1 inch). Fig. 14.13 shows timber risers and tread, jointed by tongue and grooved joints. The joints are nailed or screwed. The nosing of the step should not project beyond the face of the riser for more than the

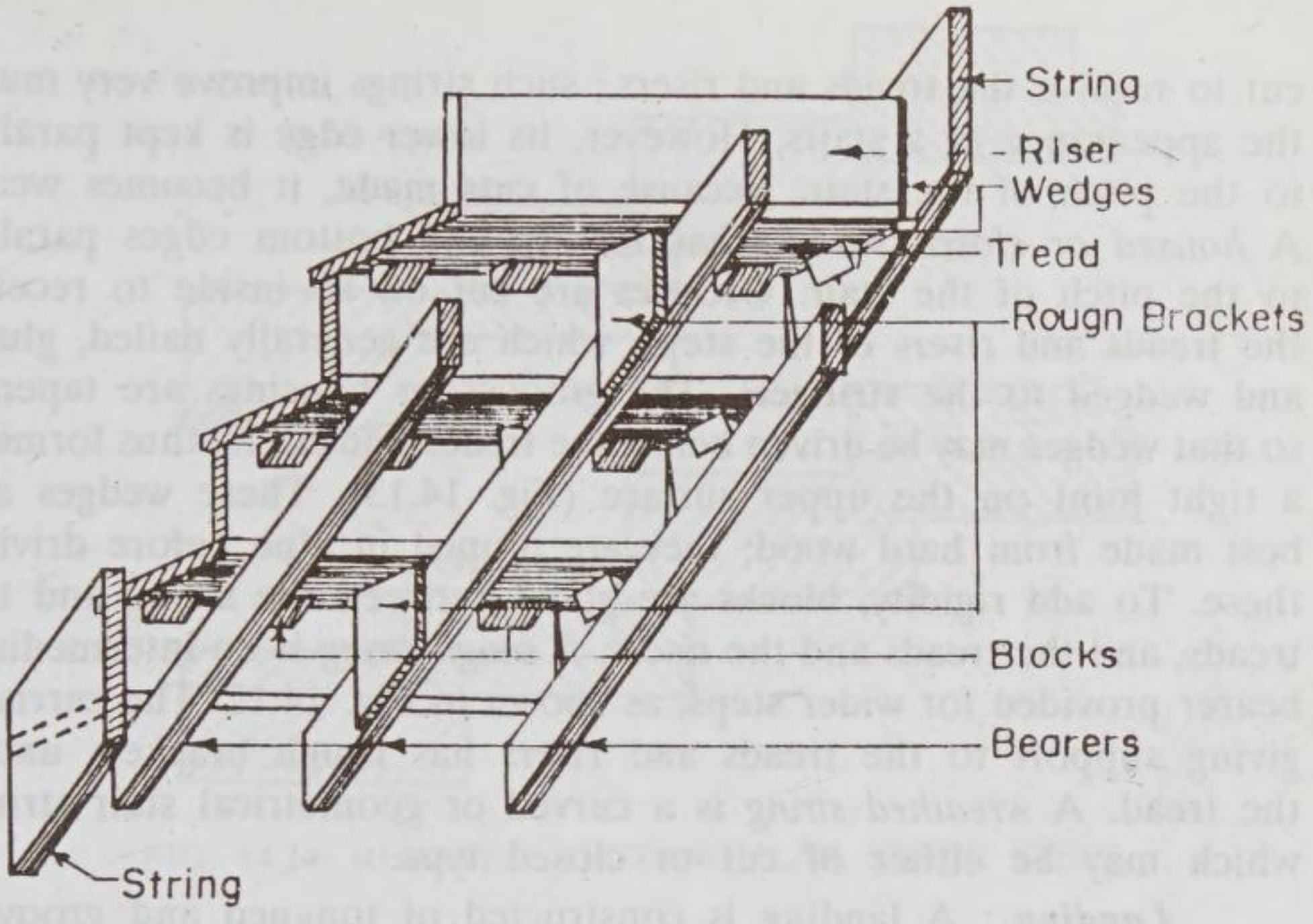


FIG. 14.12. CONSTRUCTION OF A SIMPLE TIMBER STAIRCASE.

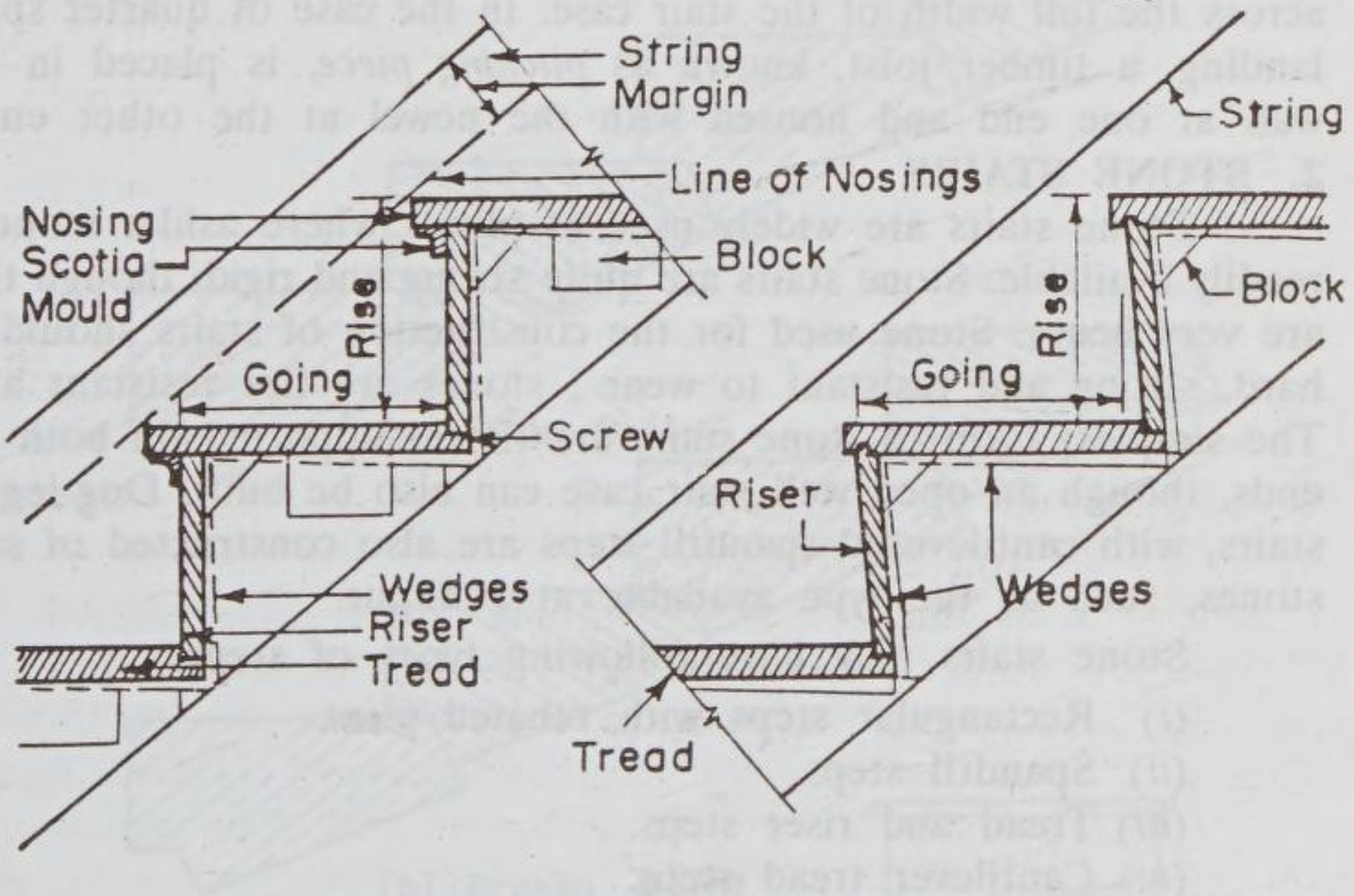


FIG. 14.13. TIMBER STAIRS DETAILS.

thickness of the tread. Scotia blocks may be provided to improve the appearance of the steps.

Stringers: These are the inclined beams of timber of 30 to 50 mm thickness and 25 to 40 cm deep, supported on newels, trimming joists or pitching pieces. These may be of four types : (*a*) cut string (*b*) housed string (*c*) rough string and (*d*) wreathed string. A *cut string* has its upper surface having carriages or houses accurately

cut to receive the treads and risers ; such strings improve very much the appearance of a stairs. However, its lower edge is kept parallel to the pitch of the stair. Because of cuts made, it becomes weak. A *housed or closed stringer* has its top and bottom edges parallel to the pitch of the stair. Grooves are cut on its inside to receive the treads and risers of the steps, which are generally nailed, glued and wedged to the stringers. The grooves or housings are tapered so that wedges may be driven below the treads and risers, thus forming a tight joint on the upper surface (Fig. 14.13). These wedges are best made from hard wood; they are dipped in glue before driving these. To add rigidity, blocks are glued between the string and the treads, and the treads and the risers. A *rough string* is an intermediate bearer provided for wider steps, as shown in Fig. 14.12. The carriage giving support to the treads and risers has rough brackets under the tread. A *wreathed string* is a curved or geometrical stair string, which may be either of cut or closed type.

Landing : A landing is constructed of tongued and grooved boarding on timber joists which are supported on walls. In the case of half space landing, a timber joist, known as *timber,* is placed across the full width of the stair case. In the case of quarter space landing, a timber joist, known as *pitching piece,* is placed in the wall at one end and housed with the newel at the other end.

2. STONE STAIRS

Stone stairs are widely used at places where ashlar stone is readily available. Stone stairs are quite strong and rigid, though they are very heavy. Stone used for the construction of stairs should be hard, strong and resistant to wear ; stones are fire resistant also. The simplest form of stone stairs are those supported on both the ends, though an open well stair case can also be built. Dog-legged stairs, with cantilevered spandril steps are also constructed of sand stones, such as the type available at Jodhpur.

Stone stairs may have following types of steps :

(*i*) Rectangular steps with rebated joint.
(*ii*) Spandril steps.
(*iii*) Tread and riser steps.
(*iv*) Cantilever tread steps.
(*v*) Built-up steps.

1. Rectangular steps

These are the simplest type, prepared from rectangular blocks of stone ashlar. The steps are arranged with the front edge of one step resting on the upper back edge of the step below, with rebated joint cut into it (Fig. 14.14).

2. Spandril steps

These steps are nearly triangular in shape so as to get a plain

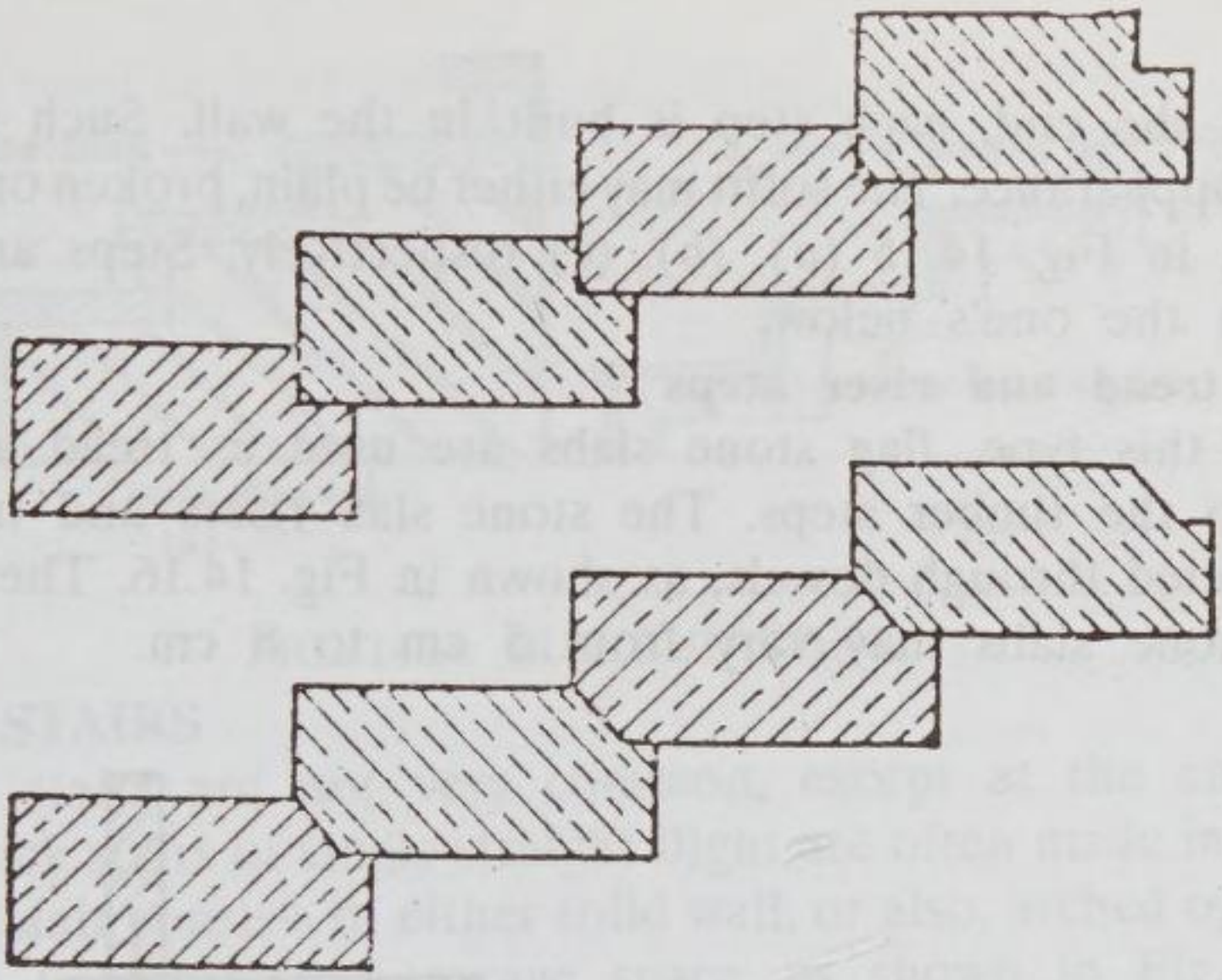


FIG. 14.14. REBATED RECTANGULAR STONE STEPS.

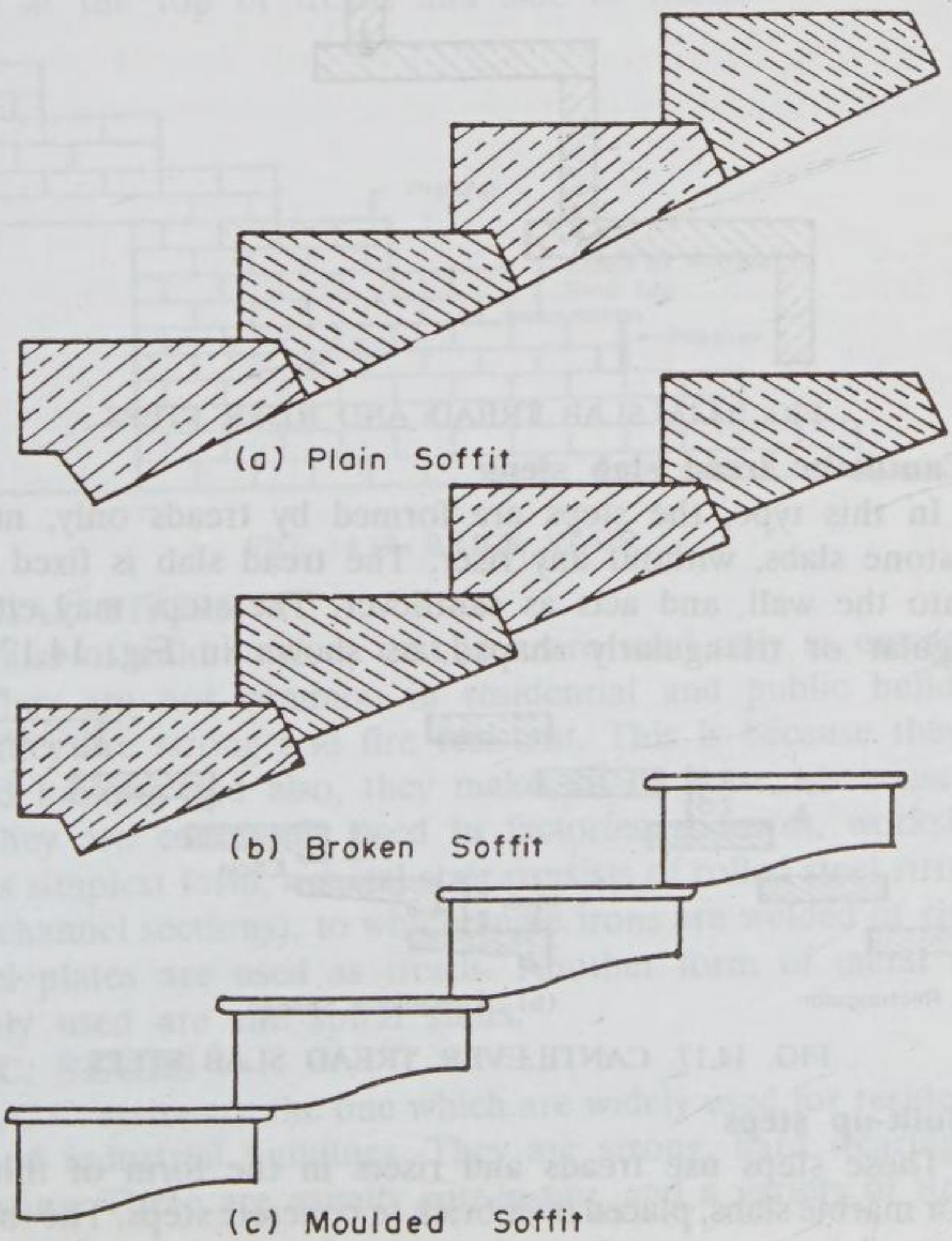


FIG. 14.15. SPANDRIL STEPS

soffit. At the end, each step is built in the wall. Such steps give pleasant appearance. The soffit may either be plain, broken or moulded, as shown in Fig. 14.15 (*a*), (*b*), (*c*) respectively. Steps are rebated to fit on the one's below.

3. Slab tread and riser steps

In this type, flag stone slabs are used as tread and risers, similar to the timber steps. The stone slab risers and treads may be connected through dowels, as shown in Fig. 14.16. The thickness of the stone slabs may vary from 5 cm to 8 cm.

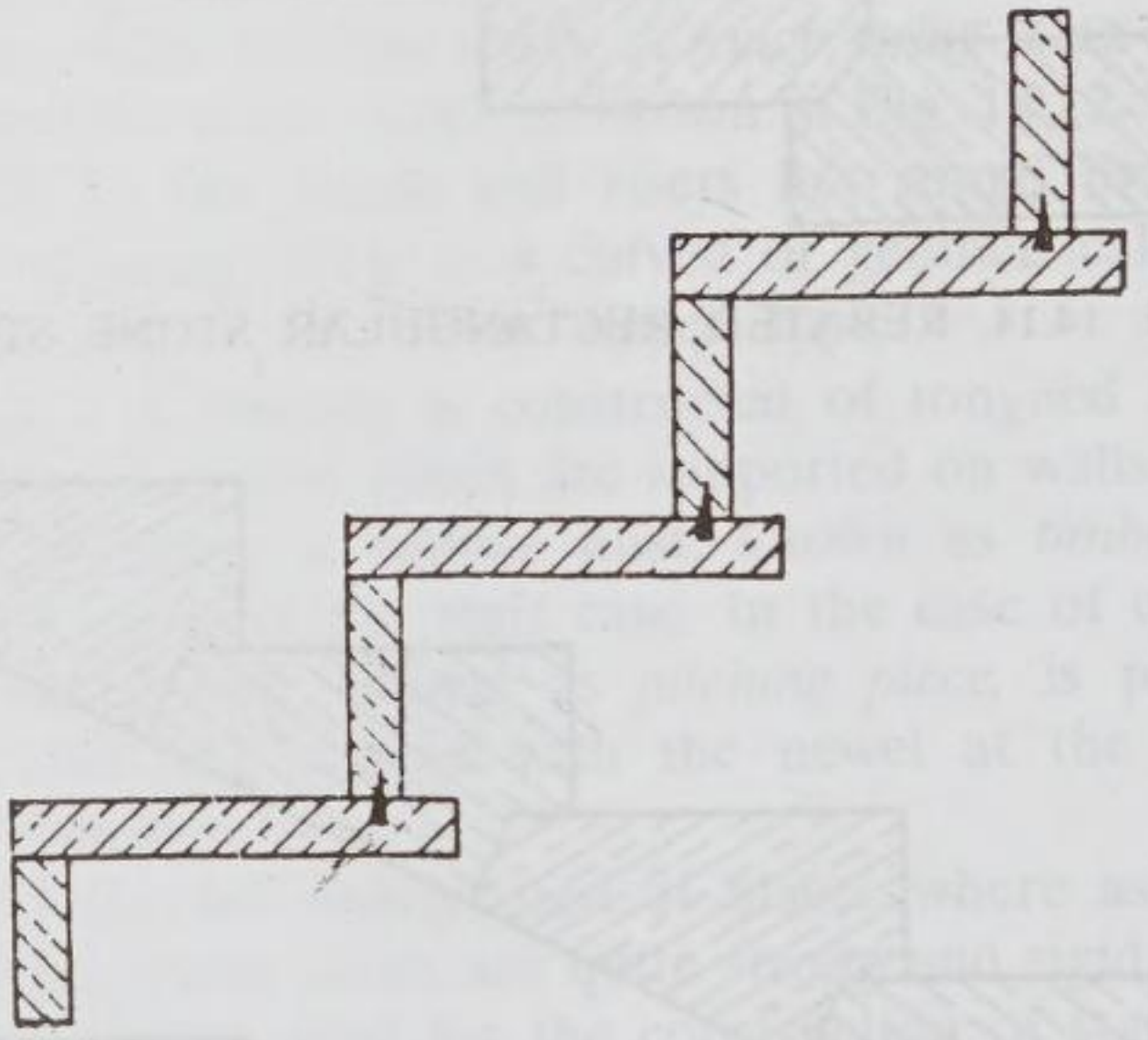


FIG. 14.16. SLAB TREAD AND RISER STEPS.

4. Cantilever tread slab steps

In this type, the steps are formed by treads only, made of thick stone slabs, without any riser. The tread slab is fixed at one end into the wall, and acts as cantilever. The steps may either be rectangular or triangularly shaped, as shown in Fig. 14.17.

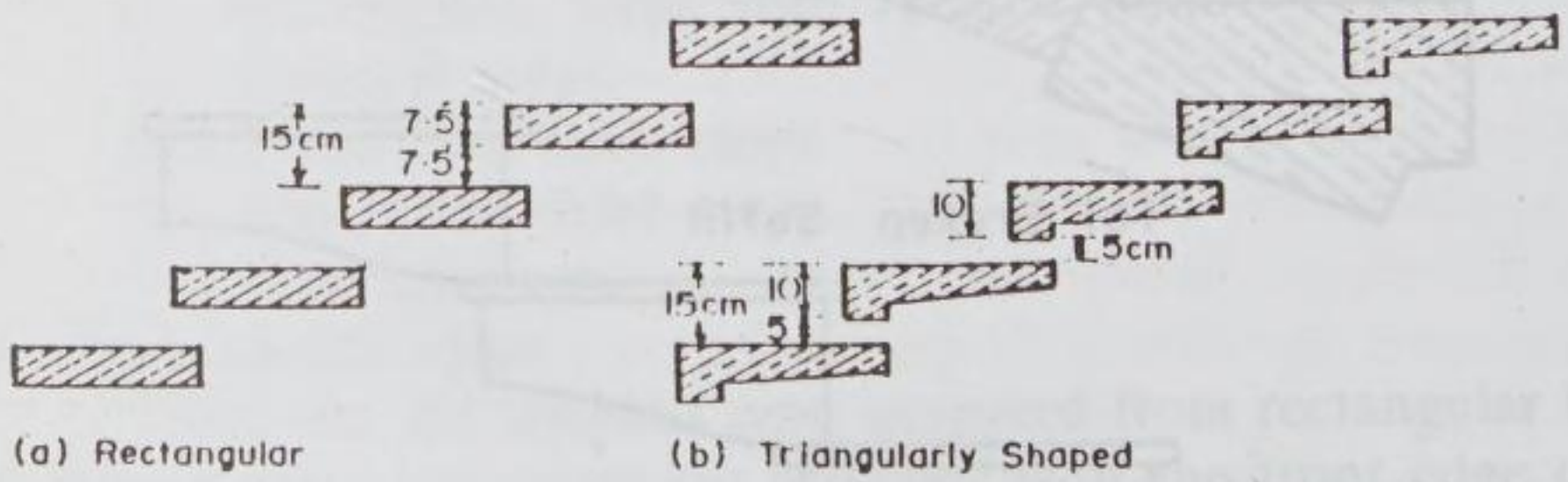


FIG. 14.17. CANTILEVER TREAD SLAB STEPS.

5. Built-up steps

These steps use treads and risers in the form of thin sawn stone or marble slabs, placed over brick or concrete steps. The thickness of stone slab may vary from 2 to 5 cm.

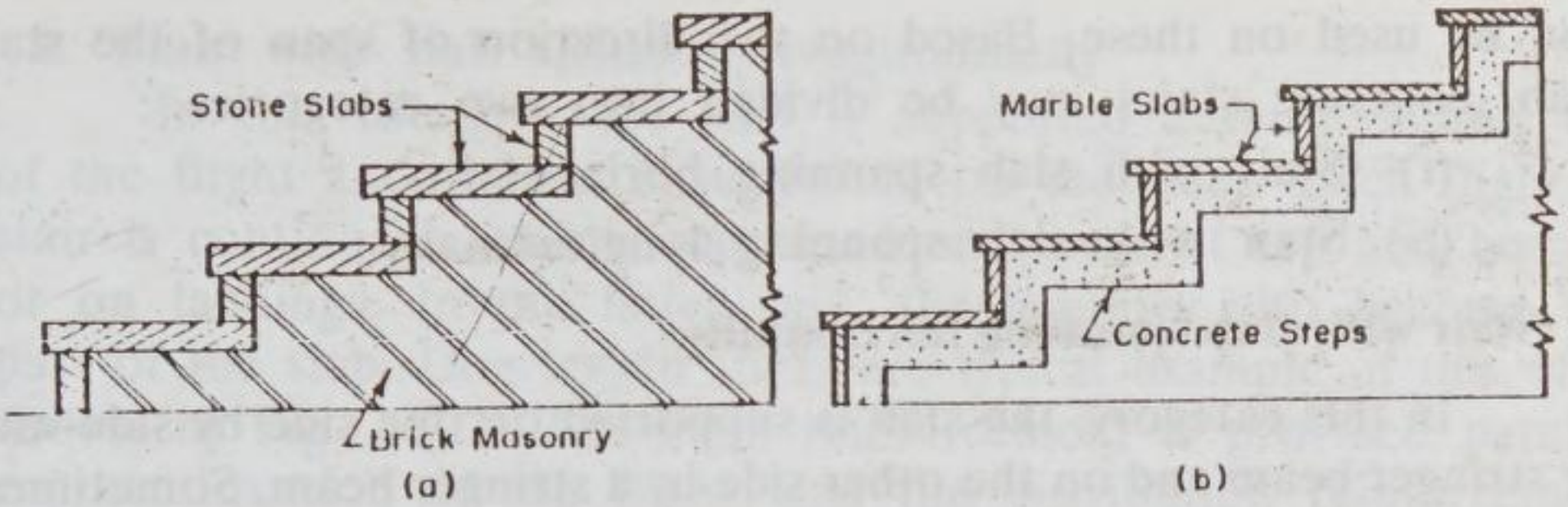


FIG. 14.18. BUILT-UP STEPS.

3. BRICK STAIRS

Brick stairs are not very common, except at the entrance. However, brick stairs of single straight flight are often made in village houses. The stair consists of either solid wall, or also, arched openings may be left for obtaining storage space, as shown in Fig. 14.19. The brick steps need frequent maintenance. Hence these may be *faced* with stone slabs. Alternatively, these steps may be cement-plastered at the top of treads and side of risers.

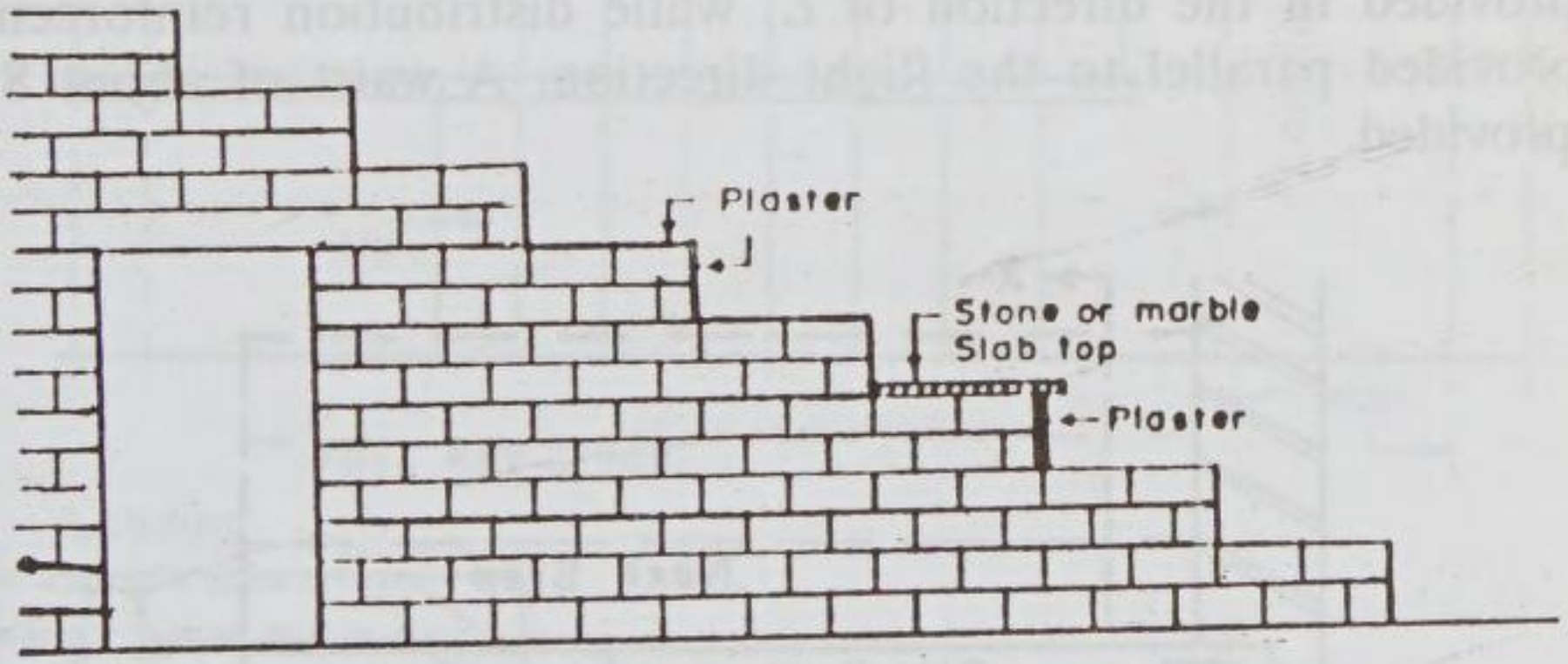


FIG. 14.19. BRICK STAIR.

4. METAL STAIRS

Stairs of mild steel or cast iron are used only as emergency stairs. They are not common in residential and public buildings, though they are strong and fire resistant. This is because they are not good looking and also, they make lot of noise when used by users. They are commonly used in factories godowns, workshops, etc. In its simplest form, a metal stair consists of rolled steel stringers (mostly channel sections), to which angle irons are welded or riveted and steel plates are used as treads. Another form of metal stairs commonly used are the spiral stairs.

5. R.C.C. STAIRS

R.C.C. stairs are the one which are widely used for residential, public and industrial buildings. They are strong, hard wearing and fire resisting. These are usually *cast-in-situ,* and a variety of finishes

can be used on these. Based on the direction of span of the stair slab, concrete stairs may be divided into two categories :

(*i*) Stair with slab spanning horizontally.

(*ii*) Stair with slab spanning longitudinally.

1. Stair with slab spanning horizontally

In this category, the slab is supported on one side by side-wall or stringer beam and on the other side by a stringer beam. Sometimes, as in the case of straight stair, the slab may be supported horizontally by side-wall on one side of each flight and the common newel on the other side between backward and forward flights. In such a case the effective span L is the horizontal distance between centre to centre of the supports. Each step is designed as spanning horizontally with the bending moment equal to $WL^2/8$, where W is the uniformly distributed load, per unit area, on the step, inclusive of the self weight. Each step is considered equivalent to a rectangular beam of width b (measured parallel to the slope of the stair) and an effective depth equal to $D/2$ as shown in Fig. 14.20. Main reinforcement is provided in the direction of L, while distribution reinforcement is provided parallel to the flight direction. A waist of about 8 cm is provided.

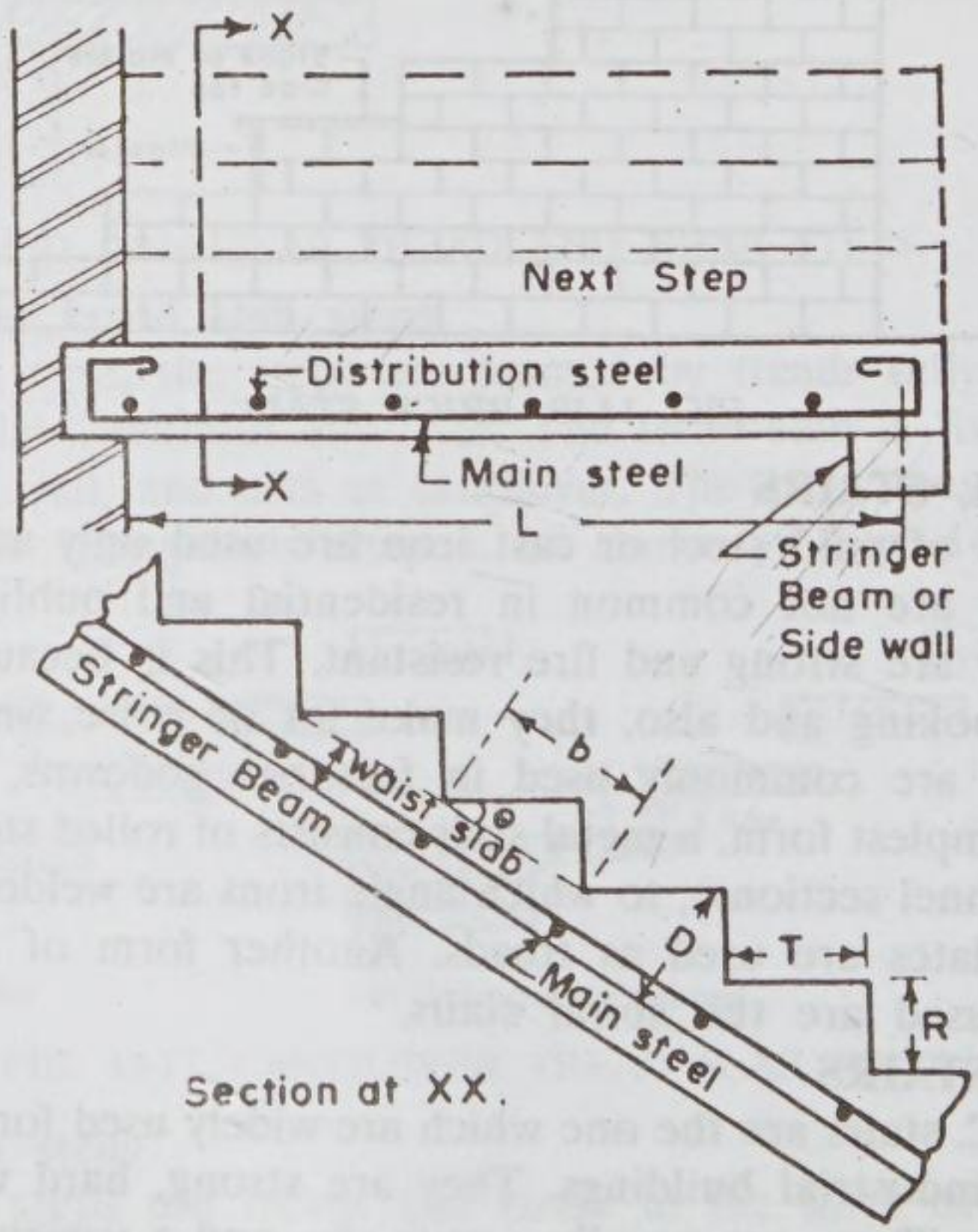


FIG. 14.20. STAIR SLAB SPANNING HORIZONTALLY.

2. Stair with slab spanning longitudinally

In this category, the slab is supported at bottom and top of the flight and remain unsupported on the sides. Each flight of stair is continuous, and is supported on beams at top and bottom or on landings. In the latter case, the landings also become the part of the slab. Dog legged stairs are typical example of this type, shown in Fig. 14.21. The main reinforcement is provided parallel to the direction of the flight, and the distribution reinforcement is provided along the width of the slab.

Sometimes, specially for wider stairs, a central stringer beam, spanning between the end walls or columns is provided on which the stairs slab (waist slab) is supported; the waist slab is designed as slab cantilevering both the sides of the stringer beam. The stringer beam itself is designed as a T-beam (Fig. 14.22).

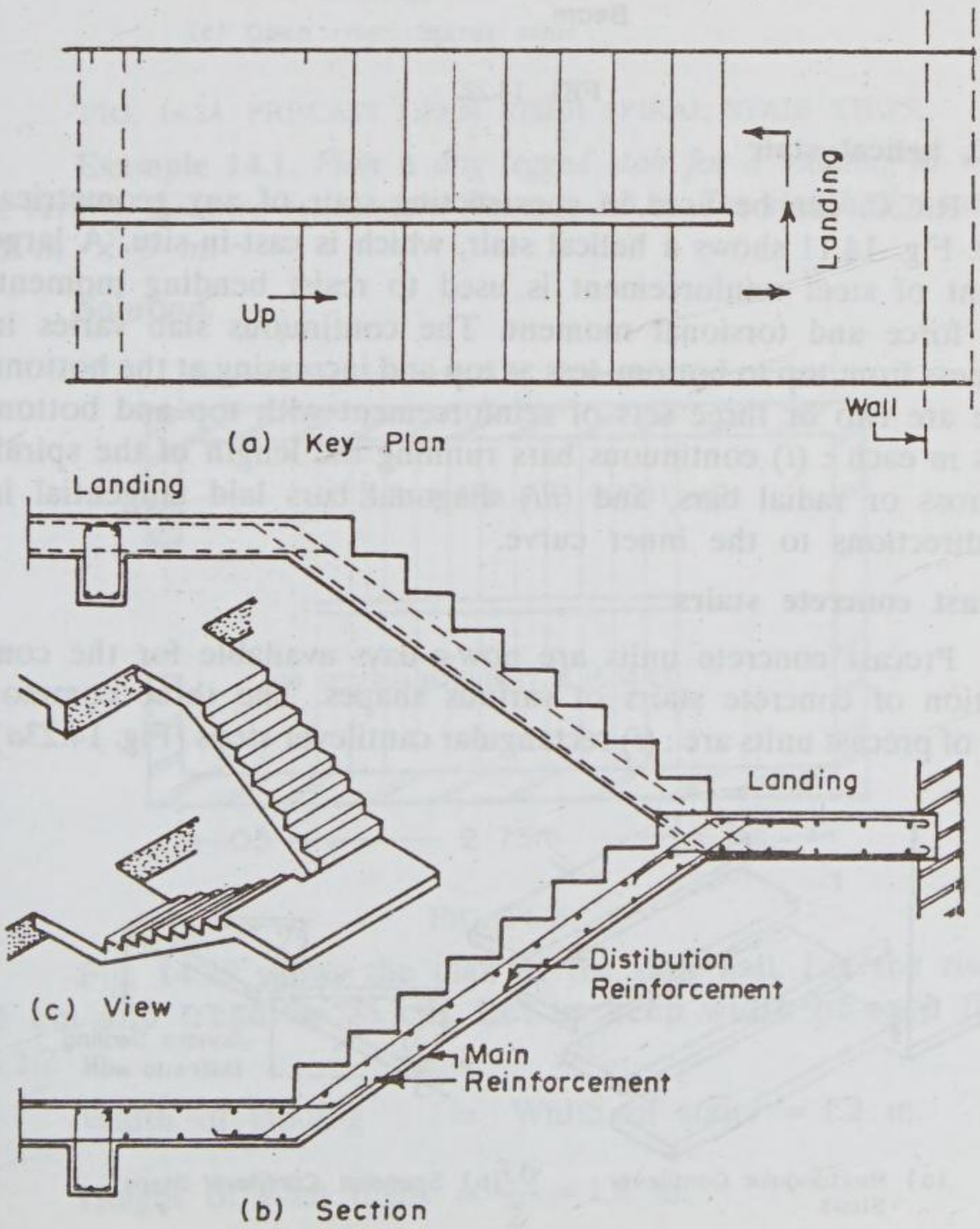


FIG. 14.21. R.C.C. DOG LEGGED STAIR.

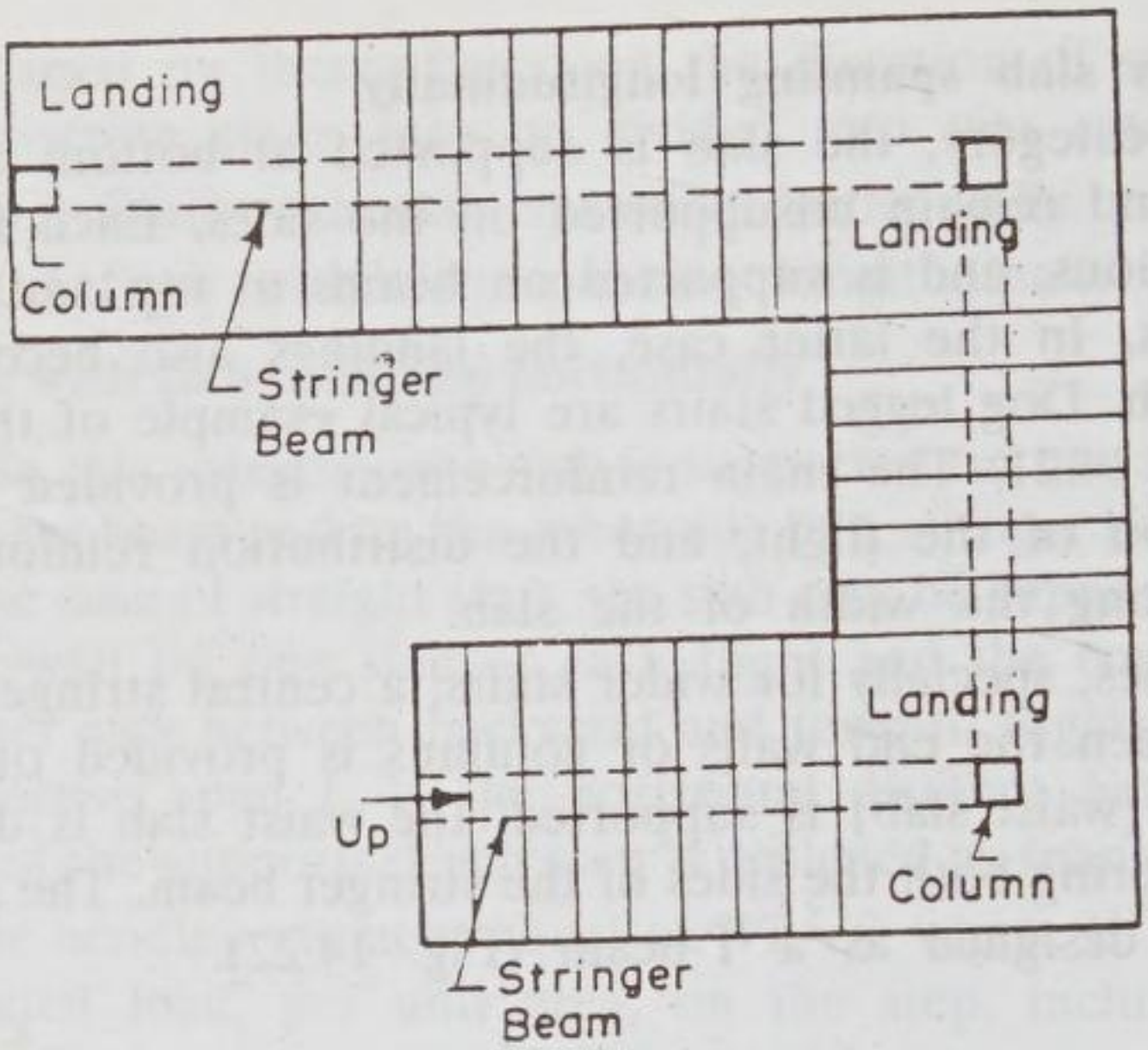


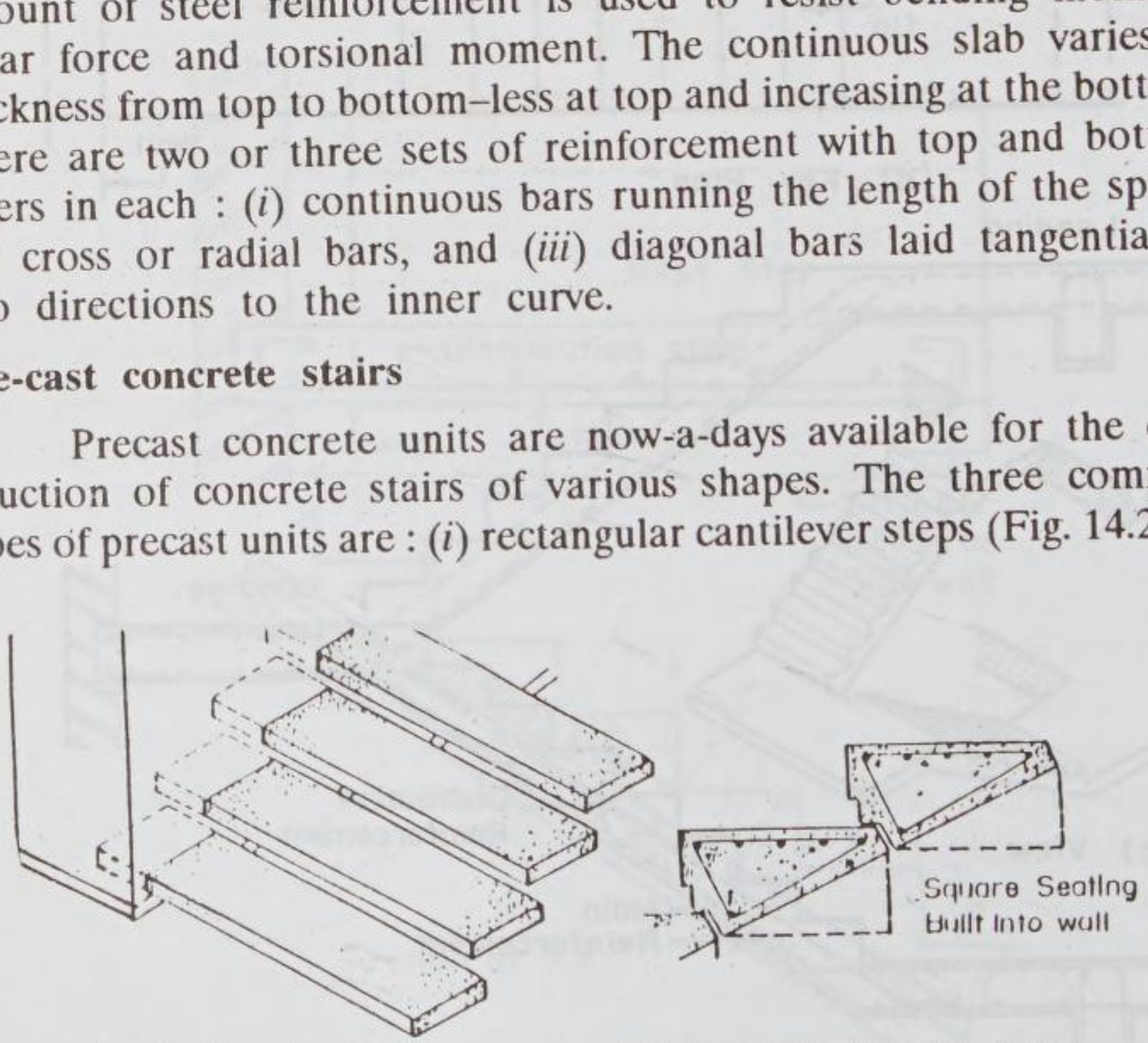
FIG. 14.22.

R.C.C. helical stair

R.C.C. can be used in constructing stair of any geometrical shape. Fig. 14.11 shows a helical stair, which is cast-in-situ. A large amount of steel reinforcement is used to resist bending moment, shear force and torsional moment. The continuous slab varies in thickness from top to bottom–less at top and increasing at the bottom. There are two or three sets of reinforcement with top and bottom layers in each : (*i*) continuous bars running the length of the spiral, (*ii*) cross or radial bars, and (*iii*) diagonal bars laid tangential in two directions to the inner curve.

Pre-cast concrete stairs

Precast concrete units are now-a-days available for the construction of concrete stairs of various shapes. The three common types of precast units are : (*i*) rectangular cantilever steps (Fig. 14.23*a*),

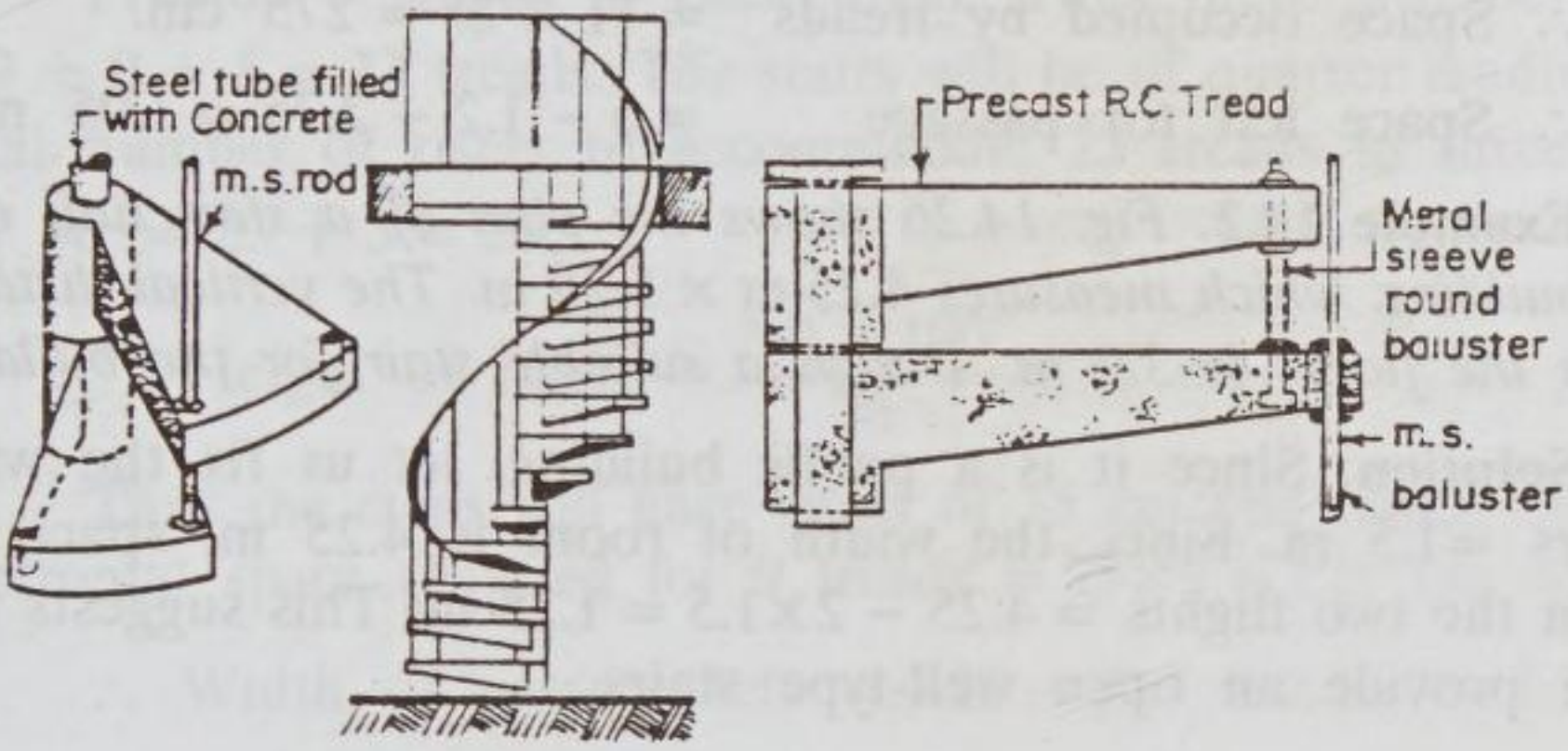


(a) Rectangular Cantilever Steps.

(b) Spandrel Cantilever Steps.

FIG. 14.23. PRECAST CONCRETE STEPS.

(*ii*) spandril cantilever steps (Fig. 14.23 *b*), and (*iii*) sector-shaped cantilever units. The latter type is used for the construction of open-riser spiral stair shown in Fig. 14.24.



(c) Open riser Spiral stair

FIG. 14.24. PRECAST OPEN RISER SPIRAL STAIR STEPS.

Example 14.1. *Plan a dog legged stair for a building in which the vertical distance between the floors is 3.6 m. The stair hall measures 2.5 m × 5 m.*

Solution.

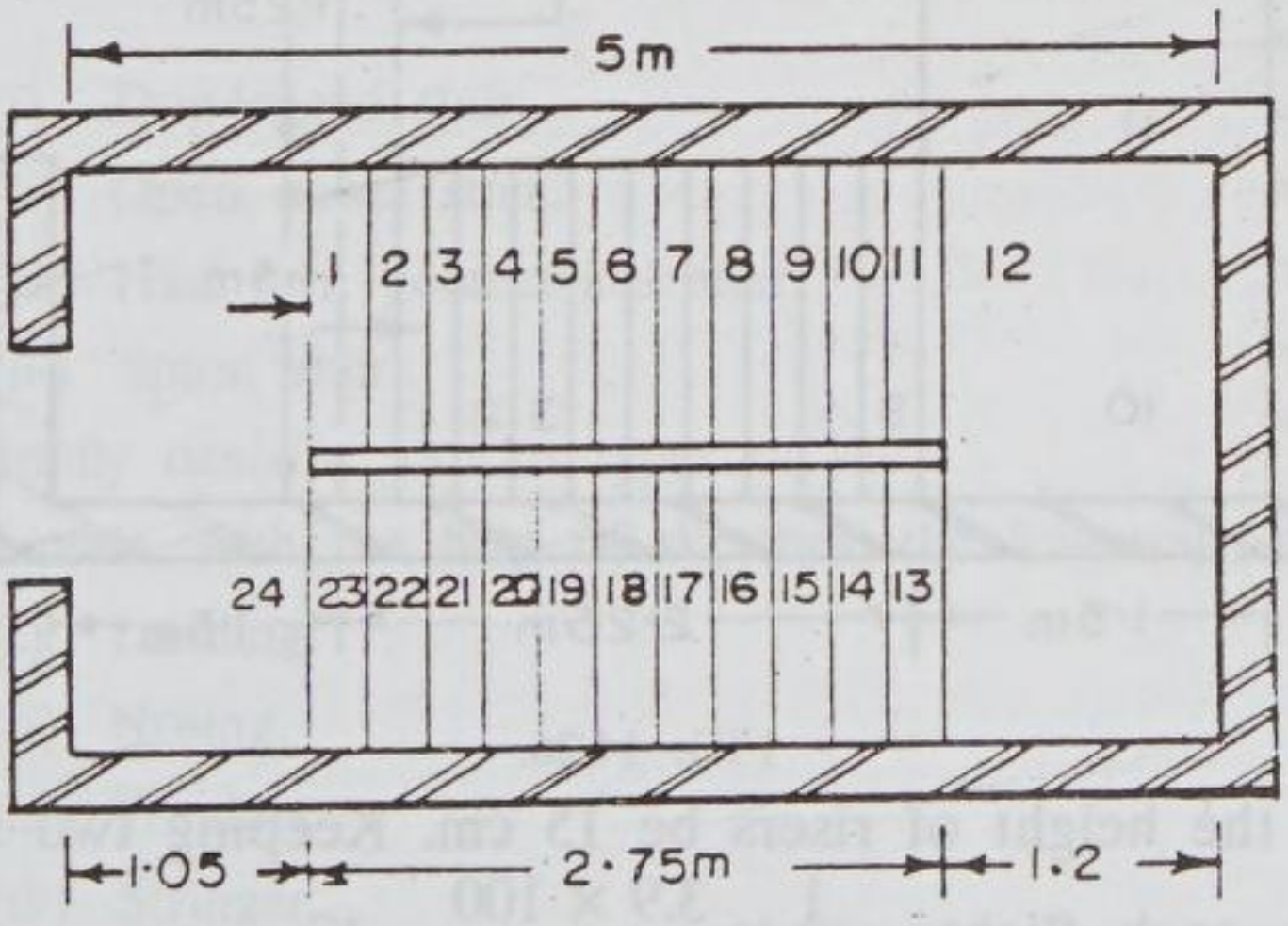


FIG. 14.25.

Fig. 14.25 shows the plan of the stair hall. Let the rise be 15 cm and tread be 25 cm. Let us keep width of each flight = 1.2 m.

Width of landing = Width of stairs = 1.2 m.

Height of each flight $= \frac{3.6}{2} = 1.8$ m.

$\therefore$ No. of risers required $= \frac{180}{15} = 12$ in each flight.

No. of treads in each flight $= 12 - 1 = 11$

$\therefore$ Space occupied by treads $= 11 \times 25 = 275$ cm.

$\therefore$ Space left for passage $= 5 - 1.2 - 2.75 = 1.05$ m.

Example 14.2. *Fig. 14.26 shows the plan of a stair hall of a public building, which measures* 4.25 m × 5.25 *m. The vertical distance between the floors is 3.9 m. Design a suitable stair for the building.*

Solution. Since it is a public building, let us fix the width of stairs =1.5 m. Since the width of room is 4.25 m, space left between the two flights = 4.25 – 2×1.5 = 1.25 m. This suggests that we can provide an open well-type stairs.

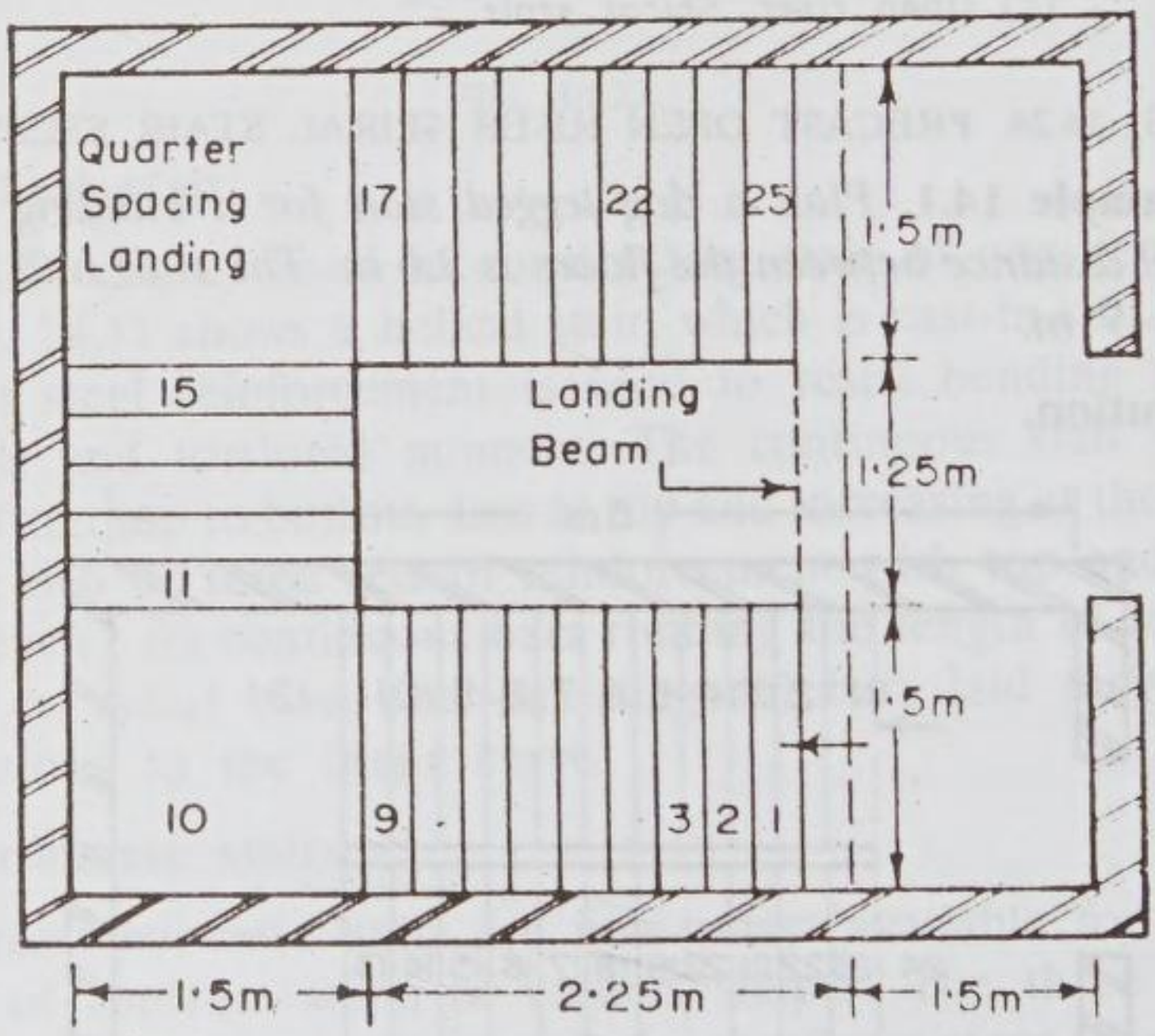


FIG. 14.26.

Let the height of risers be 15 cm. Keeping two flights, No. of riser in each flight $= \frac{1}{2} \times \frac{3.9 \times 100}{15} = 13$

$\therefore$ No. of treads in each flight $= 13 - 1 = 12$

Keeping width of tread = 25 cm, and width of landing =1.5, horizontal distance required to accommodate these =(25×12)+150 =450 cm =4.5 m. This will leave width of passage = 5.25−4.5=0.75 m only which is not sufficient. Also, in public buildings, maximum number of treads in each flight is limited to 9.

Hence let us provide 6 treads in the landing portion, which can be easily accommodated in a width $= 5 \times 25 = 125$ cm, which is equal to the width of well.

Provide 9 treads in each flight. Thus there will be a total of $9 + 9 + 5 = 23$ treads. The stairs will be of quarter landing type. Total number of risers to accommodate 23 treads in three flights will be $= 23 + 3 = 26$.

$$\text{Height of riser} = \frac{3.9 \times 100}{26} = 15 \text{ cm.}$$

Thus the steps will have risers of 15 cm and treads of 25 cm.
Horizontal space required for 9 treads $= 25 \times 9 = 225$ cm $= 2.25$ m.

$\therefore$ Width of passage left $= 5.25 - (1.5 + 2.25) = 1.50$

The plan of the stairs is shown in Fig. 14.26.

PROBLEMS

1. (*a*) State briefly the requirements of a good stair case.
   (*b*) How are the treads and risers proportioned ?
2. State the circumstances under which you use the following types of stairs :
   (*i*) Dog-legged stair.
   (*ii*) Open newel stair.
   (*iii*) Half turn geometrical stair.
   (*iv*) Spiral stair.
3. Briefly describe various types of stairs.
4. Explain, with the help of sketches, the following terms :
   (*i*) Landing.
   (*ii*) Nosing.
   (*iii*) Winders.
   (*iv*) Stringer.
   (*v*) Newel.
   (*vi*) Hand rail.
5. Write short notes on the following :
   (*i*) Metal stairs.
   (*ii*) Stone stairs.
   (*iii*) R.C.C. stairs.
   (*iv*) Spiral stairs.

(*v*) Helical stairs.

6. Draw plan and sections of a typical dog-legged stair of R.C.C.

7. Differentiate between :

(*i*) Quarter turn stair and bifurcated stair.

(*ii*) Helical stair and spiral stair.

(*iii*) R.C.C. stair with slab spanning horizontally and slab spanning longitudinally.

(*iv*) Dog-legged stair and open newel stair.

(*v*) Quarter space landing and half space landing.

8. Draw a typical sketch showing details of a timber stair. How are the risers and tread jointed ?

9. Plan a stair case for a residential building in which the vertical distance between each floor is 3.36 m. The size of the stair hall is limited to 4.5 m × 3 m.

10. Discuss the various considerations made in planning of stair cases. Illustrate the different types of stair cases generally used, indicating their suitability for specific use.

15

Roofs and Roof Coverings

15.1. INTRODUCTION

A roof may be defined as the uppermost part of the building, provided as a structural covering, to protect the building from weather (*i.e.*. from rain, sun, wind, etc.). Structurally, a roof is constructed in the same way as an upper floor, though the shape of its upper surface may be different. Basically, a roof consists of structural elements which support *roof coverings*. The structural element may be trusses, portals, beams, slabs (with or without beams), shells or domes. The *roof coverings* may be A.C. sheets, G.I. sheets, wooden shingle, tiles, slates or slab itself.

Roof and roof coverings receive rain and snow more directly and in much greater quantity than do the walls. It must, therefore, provide a *positive barrier* to the entry of rain, and vigorous weather proofing is most important. At the same time, the roof structure, which support the roof coverings must have adequate *strength* and *stability*. Apart from these, a roof must have *thermal insulation, fire resistance and sound insulation.*

Requirements of a roof

The requirements of a good roof are summarised below :

1. It should have adequate strength and stability to carry the super-imposed dead and live loads.

2. It should effectively protect the building against rain, sun, wind, etc., and it should be durable against the adverse effects of these agencies.

3. It should be water-proof, and should have efficient drainage arrangements.

4. It should provide adequate thermal insulation.

5. It should be fire resistant.

6. It should provide adequate insulation against sound. Most forms of roof construction provide for majority of buildings an adequate

insulation against sound from external sources.

15.2. TYPES OF ROOFS

Roofs may be divided into three categories :

1. Pitched or sloping roofs,
2. Flat roofs or terraced roofs, and
3. Curved roofs.

The selection of the type of roof depends upon the shape or plan of the building, climatic conditions of the area and type of constructional materials available. *Pitched roofs* have sloping top surface. These are suitable in those areas where rainfall/snowfall is very heavy. Broadly, buildings with limited width and simple shape can generally be covered satisfactorily by pitched roofs. Buildings irregular in plan, or with long spans, present awkward problems in the design of a pitched roof, involving numerous valleys, gutters and hips. Buildings of large area, such as factories, when covered by a series of parallel pitched roofs, require internal guttering in the valleys. *Flat roofs* are considered suitable for buildings in plains or in hot regions, where rainfall is moderate, and where snowfall is not there. Flat roofs are equally applicable to buildings of any shape and size. *Curved roofs* have their top surface curved. Such roofs are provided to give architectural effects. Such roofs include cylindrical and parabolic shells and shell domes, doubly curved shells such as hyperbolic paraboloids and hyperboloids of revolution, and folded slabs and prismatic shells. Such roofs are more suitable for public buildings like libraries, theatres, recreation centres etc.

15.3. PITCHED ROOFS : BASIC ELEMENTS

A roof with sloping surface is known as a pitched roof. Pitched roofs are basically of the following forms :

1. Lean-to-roof
2. Gable roof
3. Hip roof
4. Gambrel roof
5. Mansard or curb roof
6. Deck roof.

Lean-to-roof: This is the simplest type of sloping roof, provided either for a room of small span, or for the verandah. It has slope only one side (Fig. 15.1 *a*).

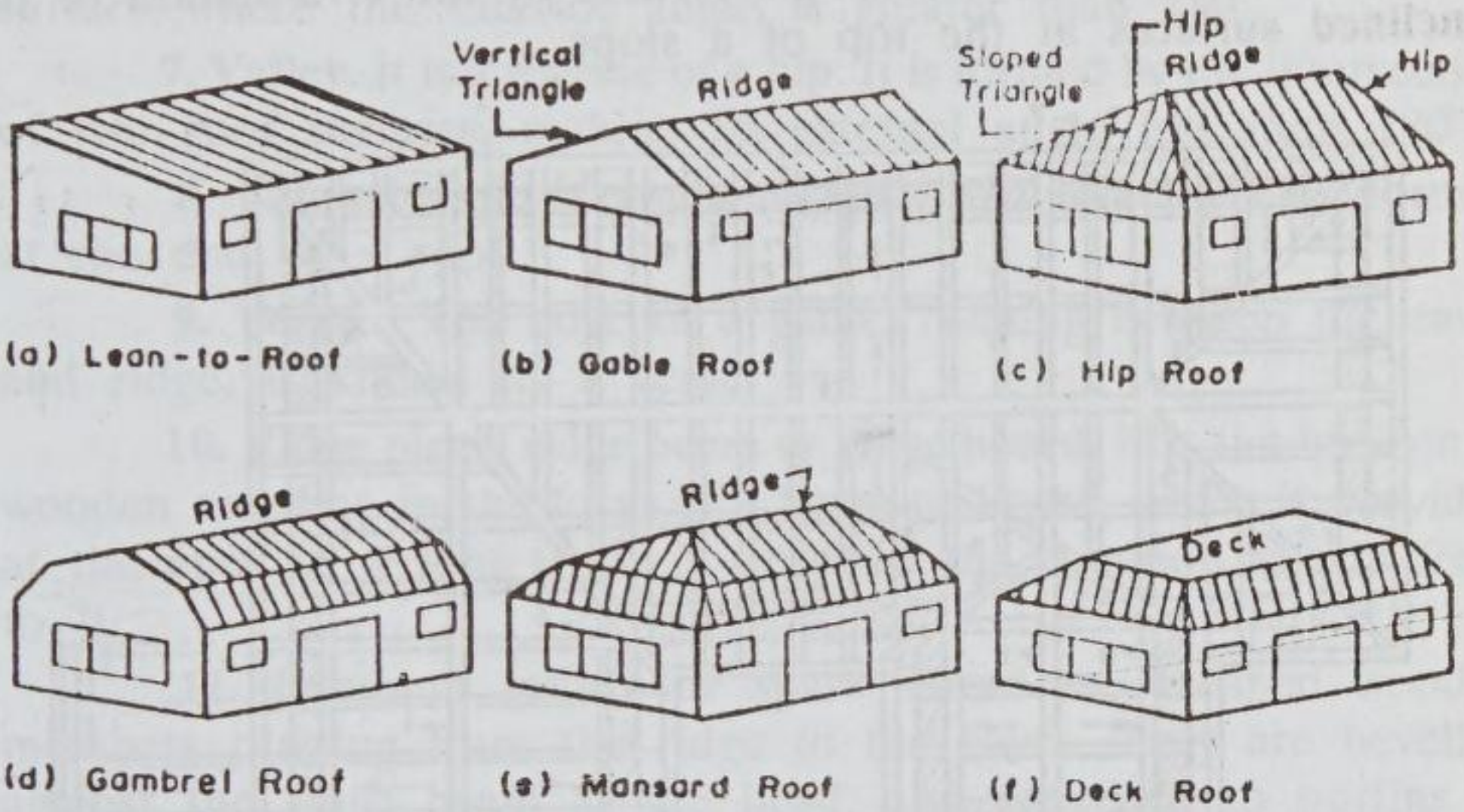
Gable roof : This is the common type of sloping roof which slopes in two directions. The two slopes meet at the ridge. At the end face, a vertical triangle if formed (Fig. 15.1 *b*).

Hip roof: This roof is formed by four sloping surfaces in four directions (Fig. 15.1*c*). At the end faces, *sloped triangles* are formed.

Gambrel roof : This roof, like gable roof, slopes in two directions, but there is a break in each slope, as shown in Fig. 15.1(*d*). At

each end, vertical face is formed.

Mansard roof : Mansard roof, like a hip roof, slopes in the four directions, but each slope has a break, as shown in Fig. 15.1(*c*). Thus, sloping ends are obtained.



15.1. VARIOUS FORMS OF SLOPING ROOFS.

Deck roof : A deck roof has slopes in all the four directions, like a hip roof, but a deck or plane surface is formed at the top, as shown in Fig. 15.1 (*f*).

Fig. 15.2 shows various elements of pitched roof. These elements are defined below :

1. **Span.** It is the clear distance between the supports of an arch, beam or roof truss.

2. **Rise.** It is the vertical distance between the top of the ridge and the wall plate.

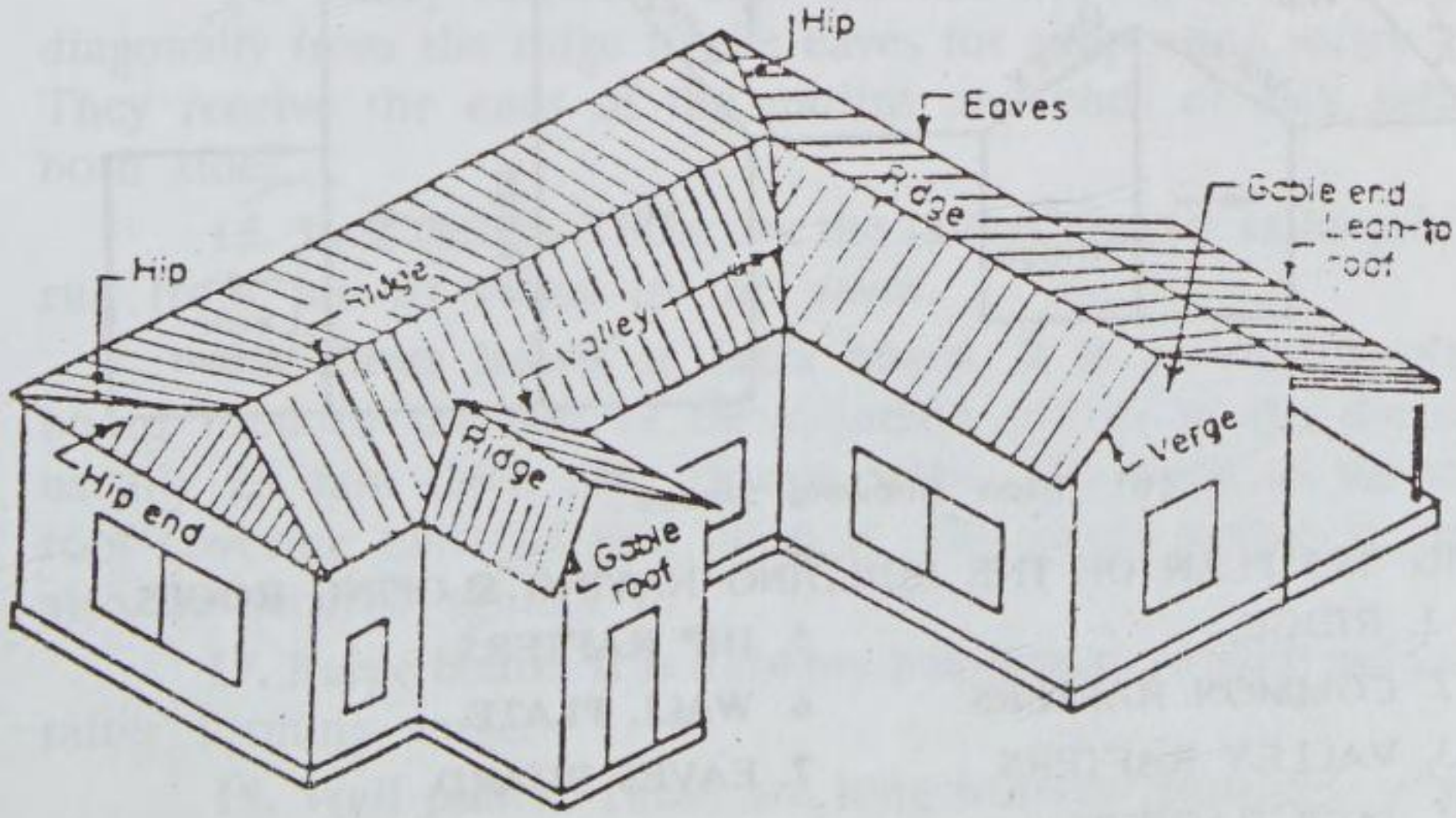
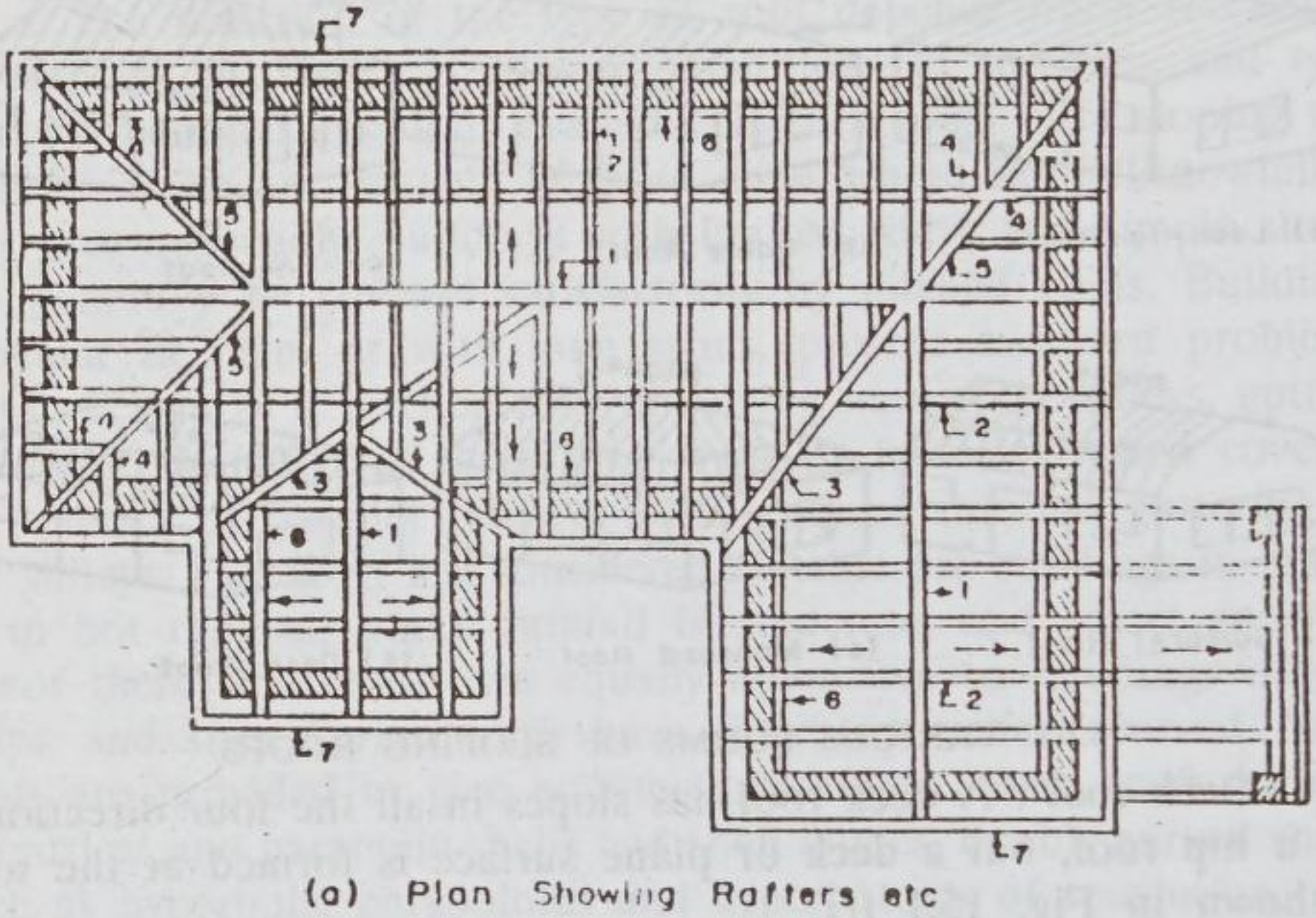


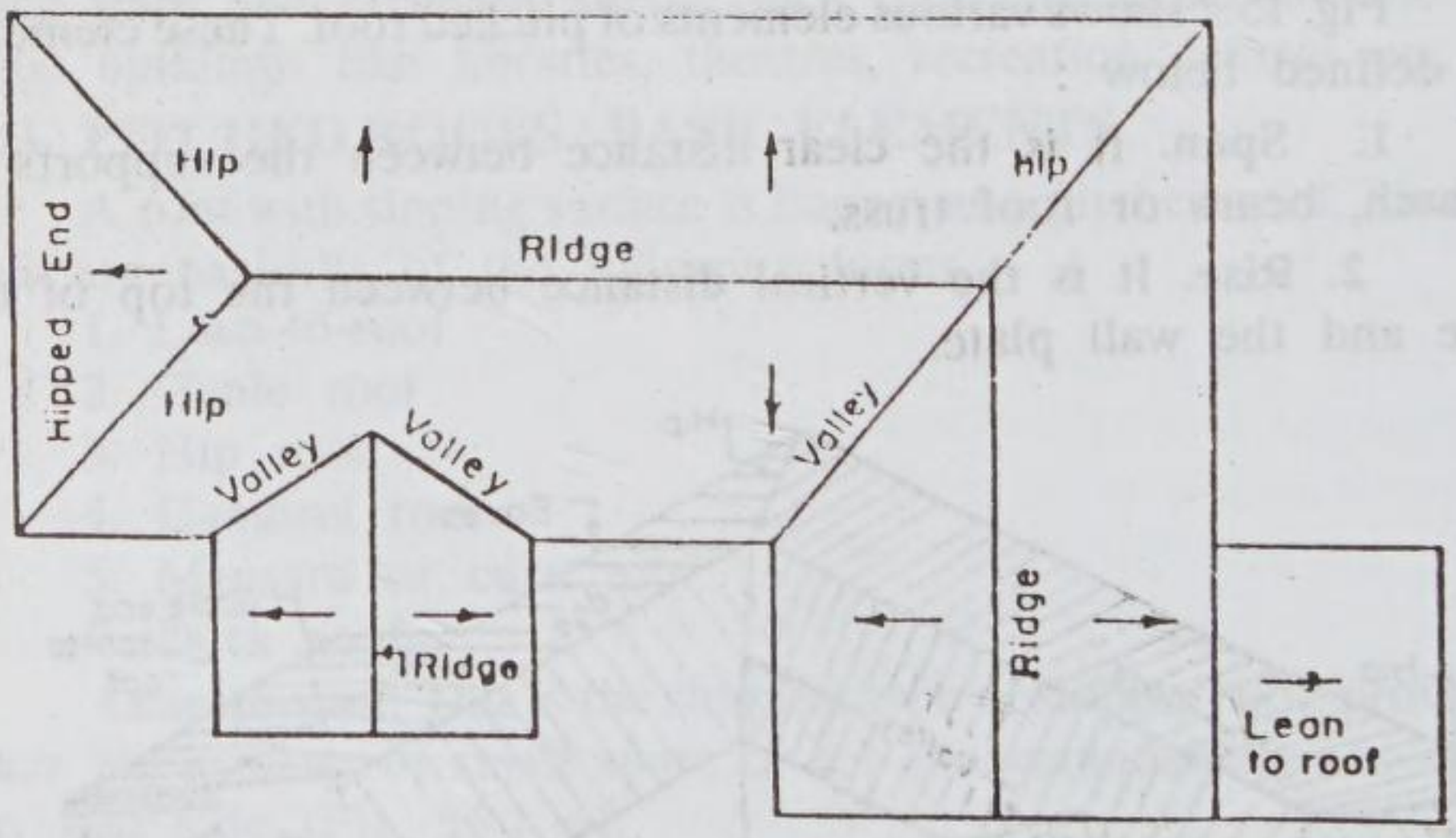
FIG. 15.2. VIEW OF A BUILDING WITH BASIC SLOPING ROOFS.

3. Pitch. It is the inclination of the sides of a roof to the horizontal plane. It is expressed either in terms of degrees (angle) or as a ratio of rise to span.

4. Ridge. It is defined as the apex line of the sloping roof. It is thus the apex of the angle formed by the termination of the inclined surfaces at the top of a slope.



(a) Plan Showing Rafters etc



(b) Plan Showing Slopes

FIG. 15.3 PLAN OF THE BUILDING HAVING SLOPING ROOFS.

1. RIDGE.
2. COMMON RAFTERS.
3. VALLEY RAFTERS.
4. JACK RAFTERS.
5. HIP RAFTERS.
6. WALL PLATE.
7. EAVES BOARD.

5. Eaves. The lower edge of the inclined roof surface is called eaves. From the lower edge (eaves), the rain water from the roof surface drops down.

6. Hip. It is the ridge formed by the intersection of two sloping surfaces,where the exterior angle is greater than 180°.

7. Valley. It is a reverse of a hip It is formed by the intersection of two roof surfaces, making an external angle less than 180°.

8. Hipped end. It is the sloped triangular surface formed at the end of a roof.

9. Verge. The edge of a gable, running between the eaves and ridge, is known as a verge.

10. Ridge piece, ridge beam or ridge board. It is the horizontal wooden member, in the form of a beam or board, which is provided at the apex of a roof truss. It supports the common rafters fixed to it.

11. Common rafters or spars These are inclined wooden members running from the ridge to the eaves. They are bevelled against the ridge beam at the head, and are fixed to purlins at intermediate point. They support the battens or boarding to support the roof coverings. Depending upon the roof covering material, the rafters are spaced 30 to 45 cm centre to centre.

12. Purlins. These are horizontal wooden or steel members, used to support common rafters of a roof when span is large. Purlins are supported on trusses or walls.

13. Hip rafters. These are the sloping rafters which form the hip of a sloped roof. They run diagonally from the ridge to the corners of the walls to support roof coverings. They receive the ends of the purlins and ends of jack rafters.

14. Valley rafters. These are the sloping rafters which run diagonally from the ridge to the eaves for supporting valley gutters. They receive the ends of the purlins and ends of jack rafters on both sides.

15. Jack rafters These are the rafters shorter in length, which run from hip or valley to the eaves.

16. Eaves board or facia board. It is a wooden plank or board fixed to the feet of the common rafters at the eaves. It is usually 25 mm thick and 25 mm wide. The ends of lower most roof covering material rest upon it. The eaves gutter, if any, can also be secured against it.

17. Barge board. It is a timber board used to hold the common rafter forming verge.

18. Wall plates. These are long wooden members, which are provided on the top of stone or brick wall, for the purpose of fixing

the feet of the common rafters. These are embedded from sides and bottom in masonry of the walls, almost at the centre of their thickness. Wall plates actually connect the walls to the roof.

19. Post plate. This is similar to a wall plate except that they run continuous, parallel to the face of wall, over the tops of the posts, and support rafters at their feet.

20. Battens. These are thin strips of wood, called scantlings, which are nailed to the rafters for lying roof materials above.

21. Boardings. They act similar to battens and are nailed to common rafter to support the roofing material.

22. Template. This is a square or rectangular block of stone or concrete placed under a beam or truss, to spread the load over a larger area of the wall.

23. Cleats. These are short sections of wood or steal (angle iron), which are fixed on the principal rafters of trusses to support the purlins.

24. Truss. A roof truss is a frame work, usually of triangles, designed to support the roof covering or ceiling over rooms.

15.4. TYPES OF PITCHED ROOFS

Pitched roofs may be broadly classified into the following :

(*a*) **Single roofs**

1. Lean-to-roof (verandah roof).
2. Couple roof.
3. Couple-close roof.
4. Collar beam roof or collar tie roof.

(*b*) **Double or purlin roofs**

(*c*) **Triple-membered or framed or trussed roofs**

1. King-post roof truss.
2. Queen-post roof truss.
3. Combination of king-post and queen-post trusses,
4. Mansard roof truss.
5. Truncated roof truss.
6. Bel-fast roof truss or latticed roof truss.
7. Composite roof trusses.
8. Steel sloping roof trusses.

Single roofs consist of only common rafters which are secured at the ridge (to ridge beam) and wall plate. These are used when span is less so that no intermediate support is required for the rafters. A *double roof* is the one in which *purlins* are introduced to support the common rafters at intermediate point. Such roofs are used when the span exceeds 5 metres. The function of a purlin is to tie the rafters together, and to act as an intermediate support to the rafters. A *triple membered or trussed roof* consists of three

sets of members : (*i*) common rafters, (*ii*) purlins, and (*iii*) trusses. The purlins, which give an intermediate support to the rafters, are themselves supported on *trusses* which are suitably spaced along the length of a room. A trussed roof is provided when the span of the room is greater than 5 metres, and when the length of the room is large, *i.e.* where there are no internal walls or partitions to support the purlins.

15.5. SINGLE ROOFS

Single roofs are those which consist of only the rafters which are supported at the ridge and at the eaves. Such roofs are used only when the span is limited to 5 metres, otherwise the size of the rafters will be uneconomical. The maximum span of the rafters is taken as 2.5 m. Single roofs are of four types : (*i*) lean-to- roof, verandah-roof or shed roof, (*ii*) couple roof, (*iii*) couple close roof, and (*iv*) collar beam roof.

1. Lean-to-roof

This is the simplest type of sloping roof, in which rafters slope to one side only. It is also known as *Pent roof or Aisle roof.* The wall to one side of the room (or verandah) is taken higher than the wall (or pillars) to the other side. A wooden wall plate is supported either on a steel corbel or a stone corbel, which are provided at 1 m centre to centre. The wall plate (or post plate) is embedded on the other side, to the wall or pillars. The difference in elevation between the two wall plates is so kept that the desired slope is obtained. Usual slope is 30°. The common rafters are nailed to wooden wall plate at their upper end, and notched and nailed to the wooden post plate at their lower end. Sometimes, iron knee straps and bolts are used to connect the rafters to the post plate. Eaves boards, battens and roof coverings are provided as shown

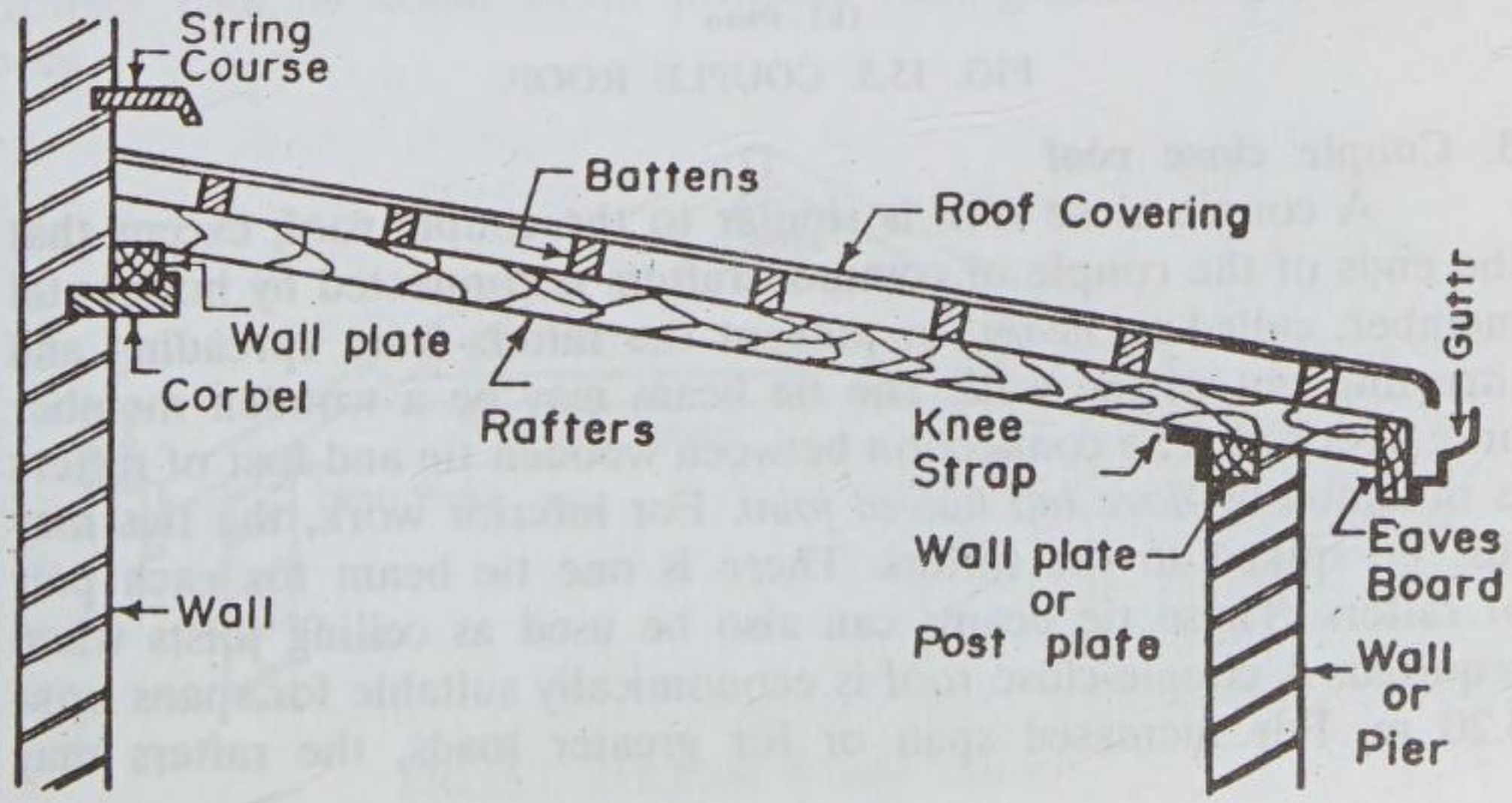


FIG. 15.4. LEAN-TO-ROOF.

in Fig. 15.4. This type of roof is suitable for maximum span of 2.5 m. These are provided for sheds, out-houses attached to main building, verandahs, etc.

2. Couple roof

This type of roof is formed by couple or pair of rafters which slope to both the sides of the ridge of the roof. The upper ends of each pair of rafter is nailed to a common ridge piece and their lower ends are notched and nailed to the wooden wall plates embedded in the masonry on the top of the outer walls. Such a roof is not very much favoured because it has the tendency to spread out at the feet (Wall plate level) and thrust out the walls supporting the wall plates. Due to this, the couple roof is used when the span is limited to 3.6 metres.

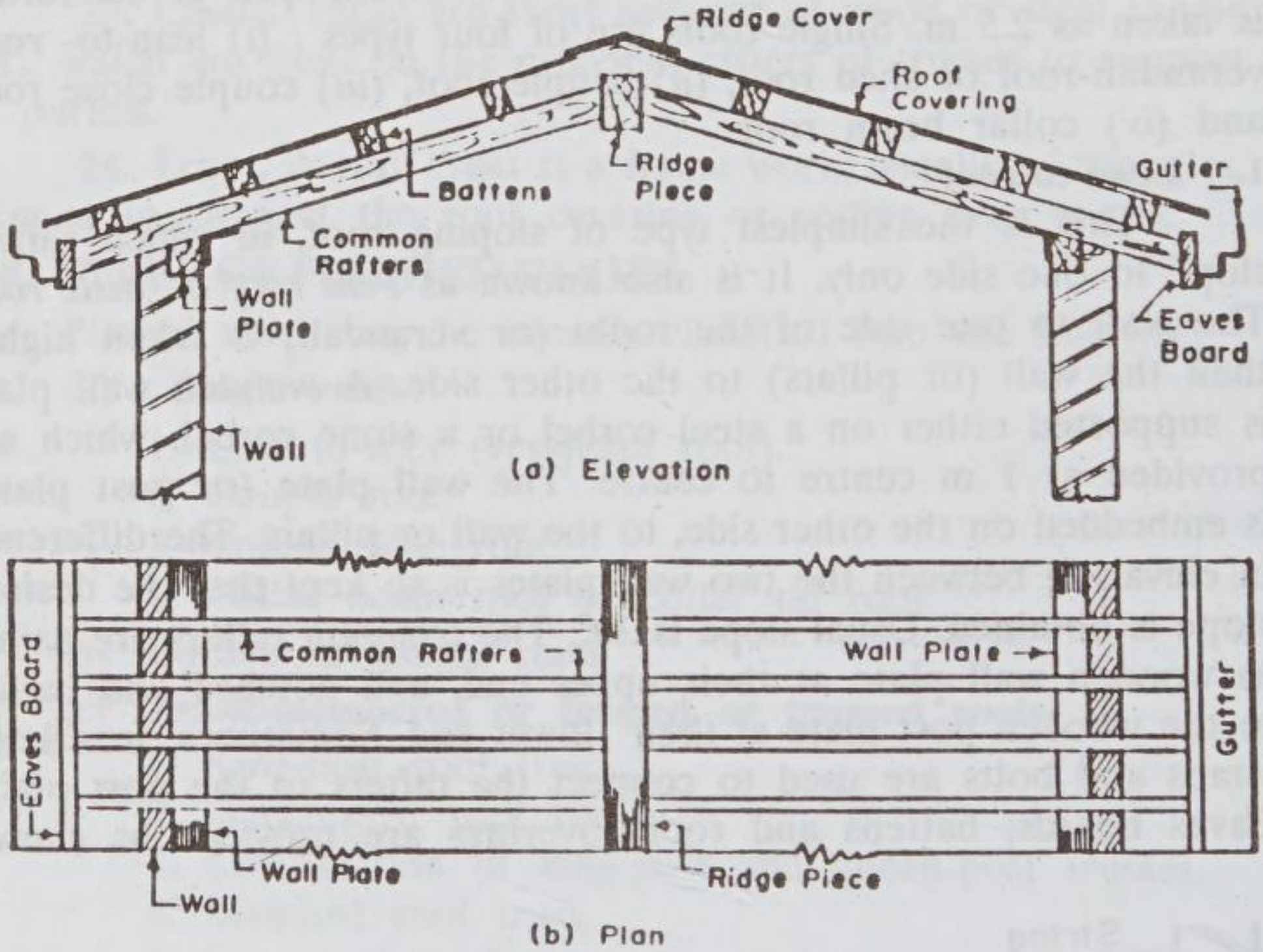


FIG. 15.5. COUPLE ROOF.

3. Couple close roof

A couple close roof is similar to the couple roof, except that the ends of the couple of common rafters is connected by horizontal member, called *tie beam,* to prevent the rafters from spreading and thrusting out of the wall. The tie beam may be a wooden member or a steel rod. The connection between wooden tie and feet of rafters is obtained by *dove tail halved joint.* For inferior work, the ties may just be spiked to the rafters. There is one tie beam for each pair of rafters. These tie beams can also be used as ceiling joists when required. A couple-close roof is economically suitable for spans upto 4.20 m. For increased span or for greater loads, the rafters may

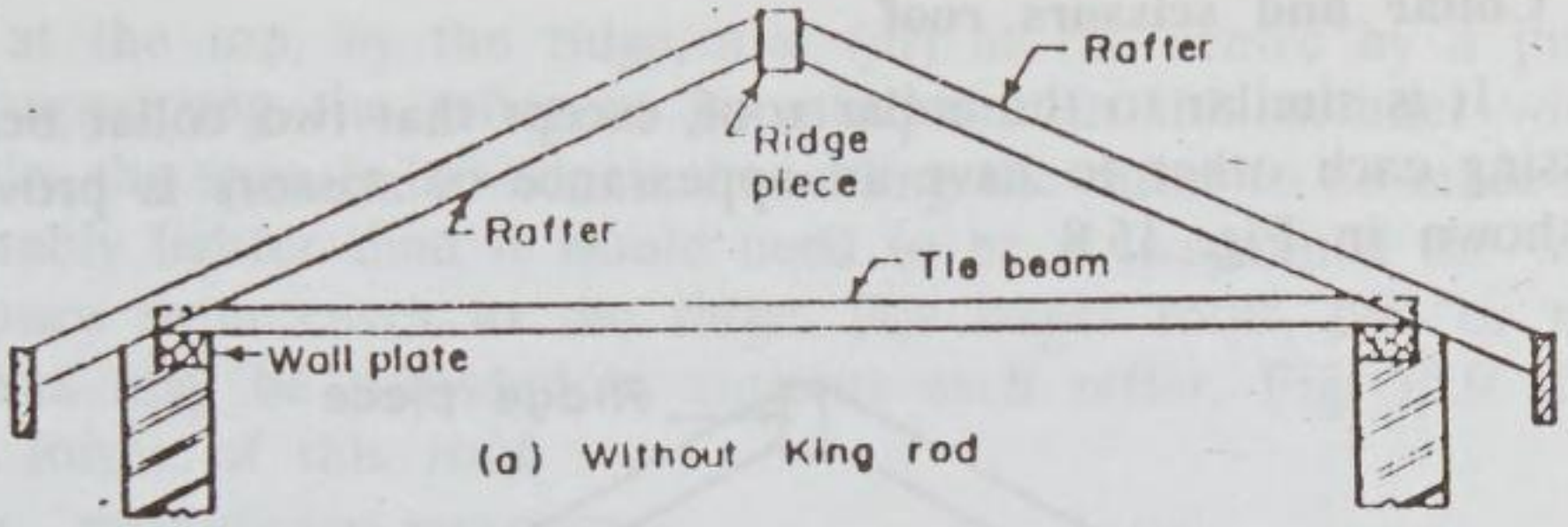


FIG. 15.6. CLOSE COUPLE ROOF.

have tendency to sag in the middle. This can be checked by providing a central vertical rod, called *king rod* or *king bolt* which connects the ridge piece and the tie beam as shown in Fig. 15.6 (*b*).

4. Collar beam roof

When the span increases, or when the load is more, the rafters of the couple close roof have the tendency to bend. This is avoided by raising the tie beam and fixing it at one-third to one-half of the vertical height from wall plate to the ridge. This raised beam is known as the *collar beam* (or *collar tie*). Thus, a collar beam roof is similar to a close couple roof, except that in the latter case a tie beam is provided at the level of wall plates while in this case a collar beam is provided at the raised level (Fig. 15.7). This roof is suitable for spans upto 5 metres. A lower collar position gives stronger roof. A collar beam provides roof greater height of the room.

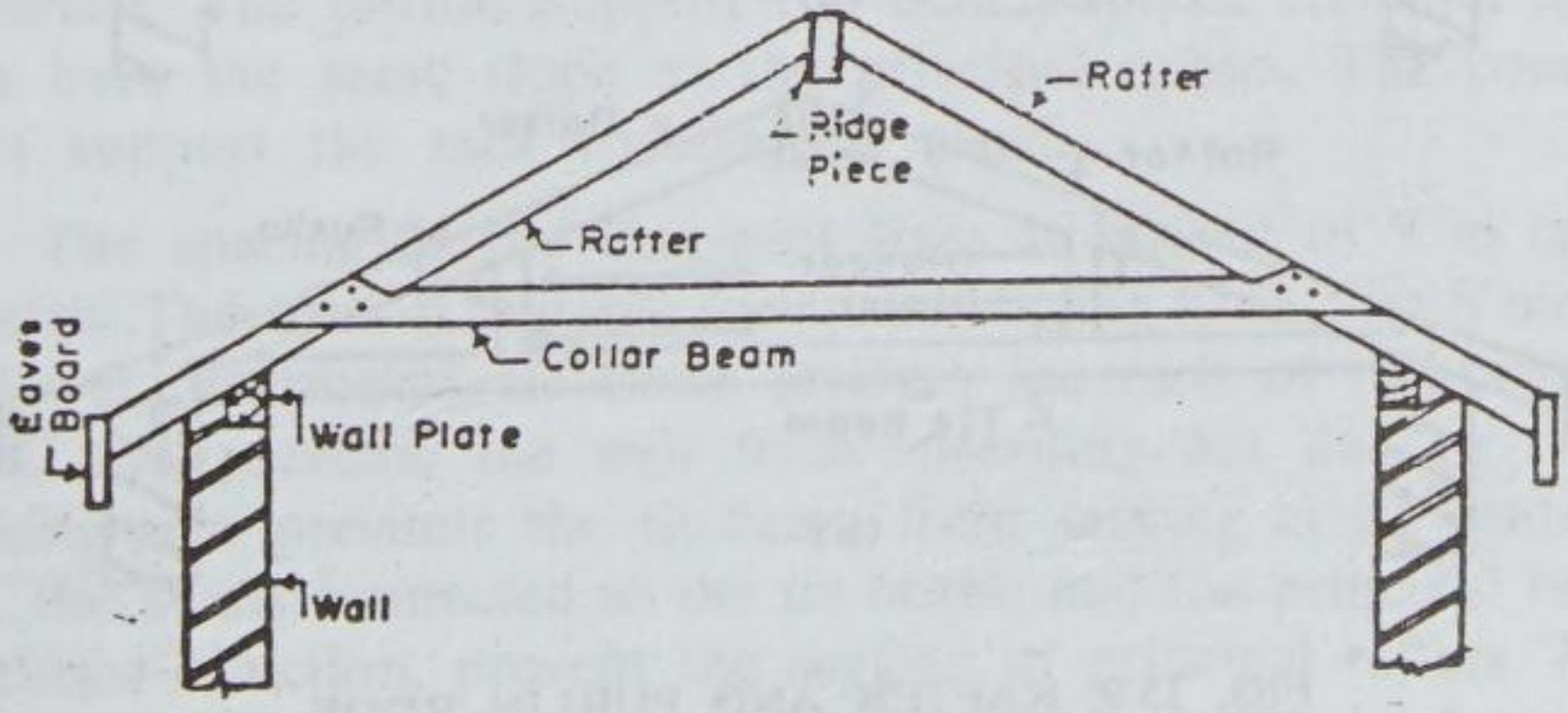


FIG.15.7. COLLAR BEAM ROOF.

5. Collar and scissors roof

It is similar to the collar roof, except that two collar beams, crossing each other to have an appearance of scissors is provided as shown in Fig. 15.8.

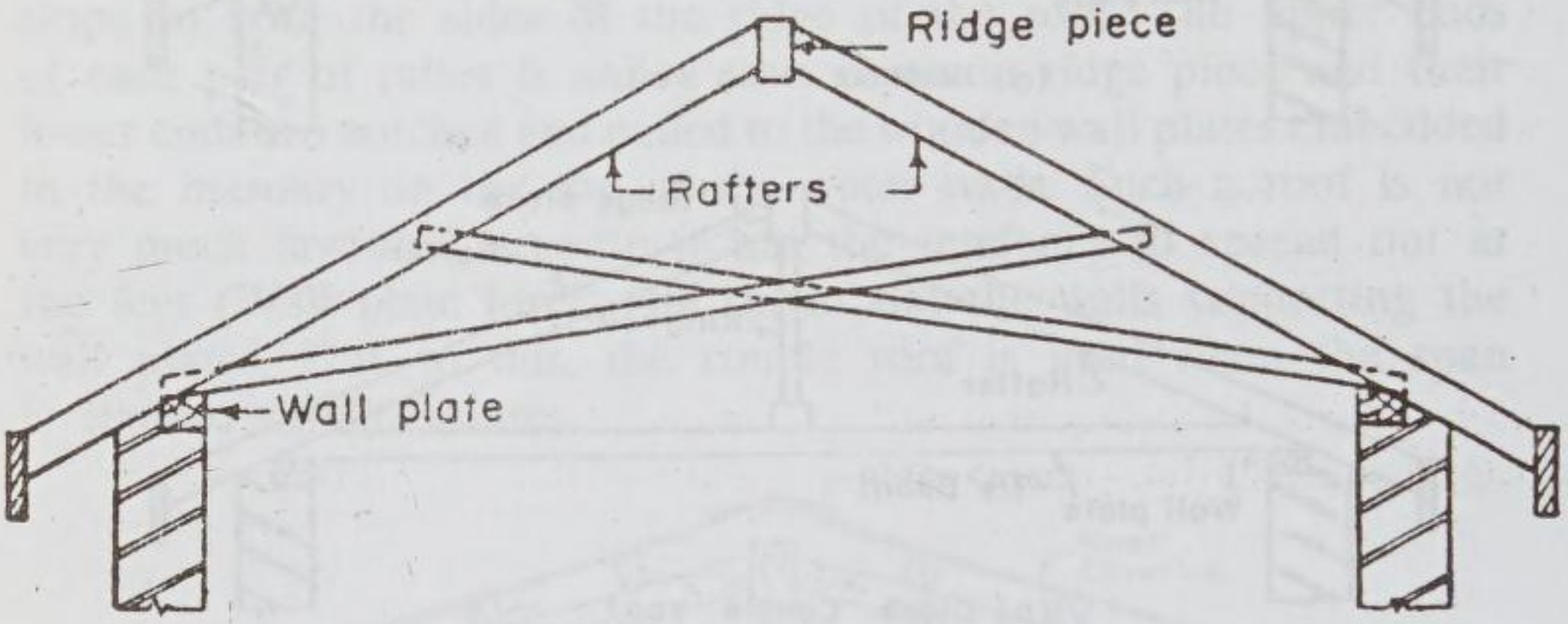


FIG. 15.8. COLLAR AND SCISSORS ROOF.

15.6. DOUBLE OR PURLIN ROOFS

These roofs have two basic elements : (*i*) rafters, and (*ii*) purlins. The purlins give intermediate support to the rafters, and are supported on end walls. The intermediate supports so provided in the form of purlins, reduce the size of the rafters to the economical range. Such a roof is also known as *rafter and purlin roof*. The rafters are provided fairly close (40 to 60 cm c/c). Each rafter is thus supported at three points : (*a*) at the bottom ; on the wall through wall plate,

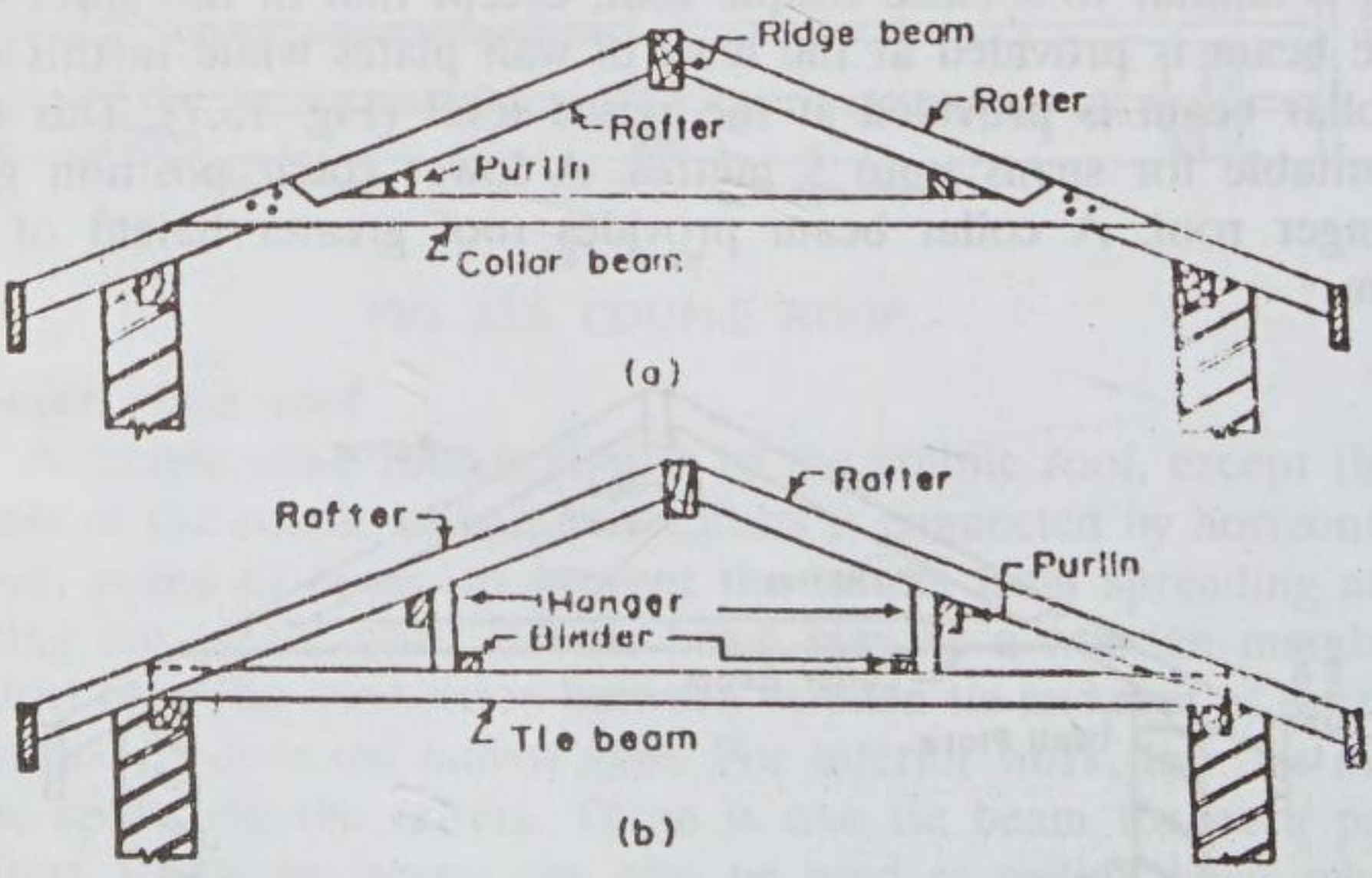


FIG. 15.9. RAFTER AND PURLIN ROOF

(*ii*) at the top, by the ridge, and (*iii*) at the centre by a purlin. By supporting the rafter at its mid-point in this manner with a purlin, the span is halved, thus enabling the rafter to be made considerably lighter than it would need to be if it spanned the whole distance from eaves to the ridge. For larger roofs, two or more purlins may be provided to support each rafter. Fig. 15.9 shows two forms of this roof.

15.7. TRUSSED ROOFS

When the span of the roof exceeds 5 m and where there are no inside walls to support the purlins, framed structures, known as *trusses* are provided at suitable interval along the length of the room. Spacing is generally limited to 3 metres for wooden trusses. In this system, the roof consists of three elements : (*i*) rafters to support the roofing material (*i.e.*. tiles etc.), (*ii*) purlins to provide intermediate support to rafters, and (*iii*) trusses to provide support to the ends of purlins. The trusses span in the same direction in which the couple of rafters run. The trusses also support the ridge piece or ridge beam. The various types of trusses in use are :

(*i*) King-post truss.
(*ii*) Queen-post truss.
(*iii*) Combination of king-post and queen-post trusses.
(*iv*) Mansard truss.
(*v*) Truncated truss.
(*vi*) Bel-fast truss.
(*vii*) Steel trusses.
(*viii*) Composite trusses.

The first six types are essentially wooden trusses.

1. King-post truss

A king-post truss, shown in Fig. 15.10 consists of the following components : (*i*) lower tie beam, (*ii*) two inclined principal rafters. (*iii*) two struts, and (*iv*) a king post. The principal rafters support the *purlins*. The purlins support the closely-spaced *common rafters* which have the same slope as the principal rafters. The common rafters support the roof covering as usual.

The spacing of the king-post truss is limited to 3 m centre to centre. The truss is suitable for spans varying from 5 to 8 metres. The lower, horizontal, tie beam receives the ends of the principal rafters, and prevents the wall from spreading out due to thrust. The king-post prevents the tie-beam from sagging at its centre of span. The struts connected to the tie beams and the principal rafters in inclined direction, prevent the sagging of principal rafters. *Ridge beam* is provided at the apex of the roof to provide end support

to the common rafters. The trusses are supported on the bed blocks of stone or concrete, embedded in the supporting walls so that load is distributed to a greater area.

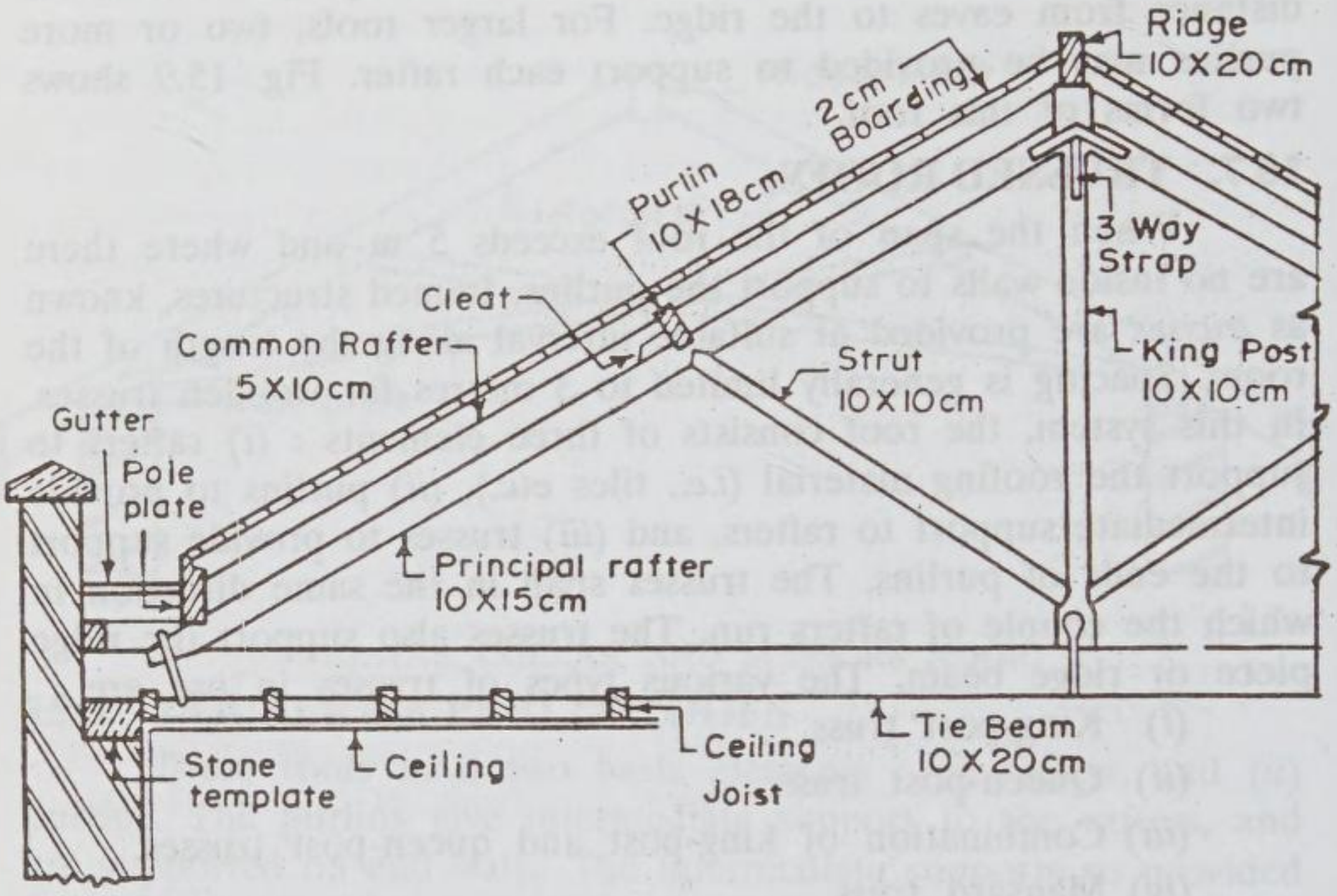


FIG. 15.10. KING-POST TRUSS (SPAN 7 M.)

The principal rafter is jointed to the tie beam by a 'single abutment and tenon joint' or by a 'bridle joint'. The joint is further strengthened by a wrought iron heel strap, would round the joint.

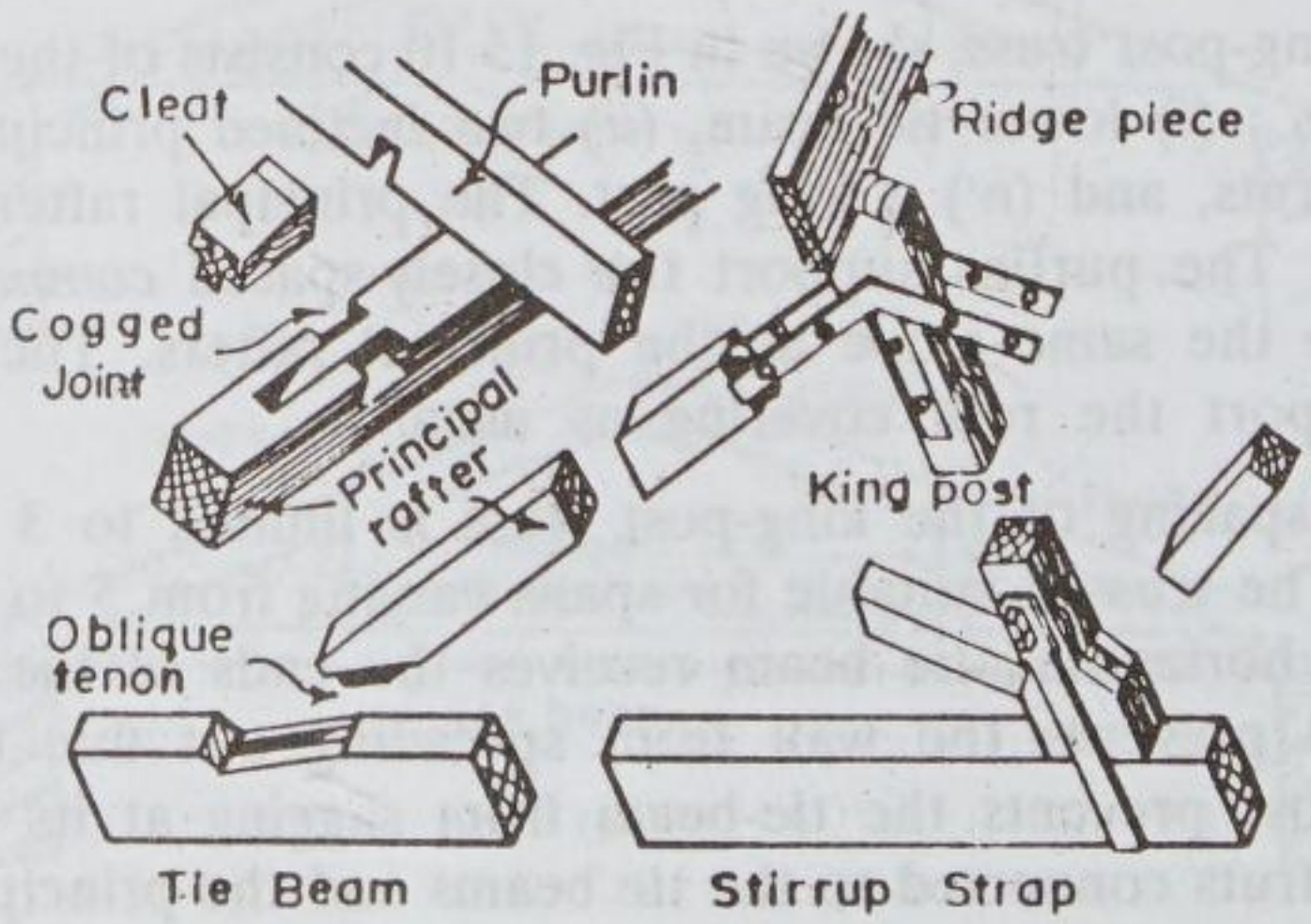


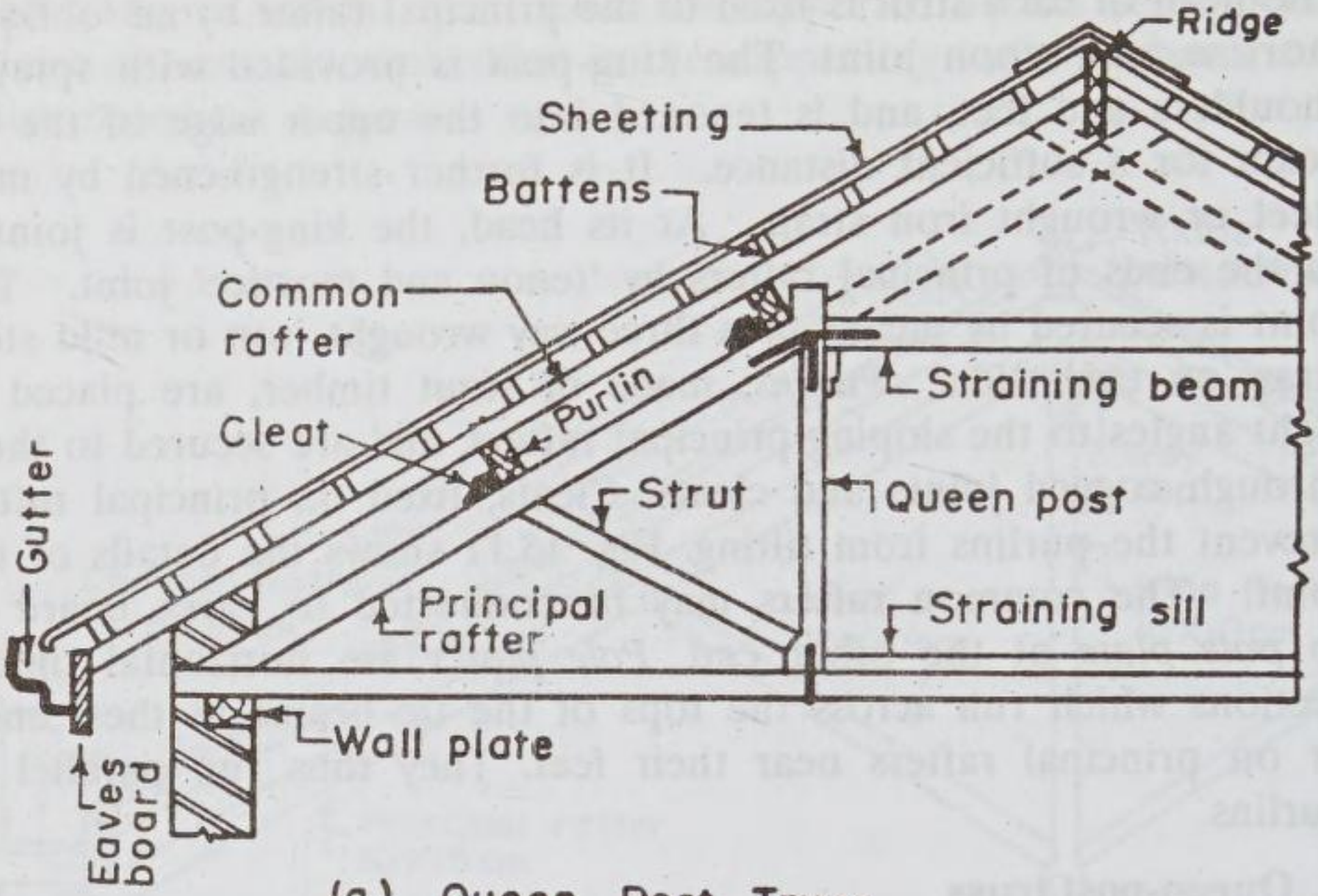
FIG. 15.11. DETAILS OF JOINTS IN KING-POST TRUSS.

The head of each strut is fixed to the principal rafter by an 'oblique' mortise and tenon joint. The king-post is provided with splayed shoulders and feet, and is tenoned into the upper edge of the tie beam for a sufficient distance. It is further strengthened by mild steel or wrought iron strap. At its head, the king-post is jointed to the ends of principal rafters by 'tenon and mortice' joint. The joint is secured by means of a three-way wrought iron or mild steel strap on each side. *Purlins,* made of stout timber, are placed at right angles to the sloping principal rafters, and are secured to them through cogged joints and cleats. Cleats, fixed on principal rafter, prevent the purlins from tilting. Fig. 15.11 shows the details of the joint. The common rafters may be connected to eaves board or to *pole plate* at the other end. *Pole plates* are horizontal timber sections which run across the tops of the tie beams at their ends, or on principal rafters near their feet. They thus run parallel to purlins.

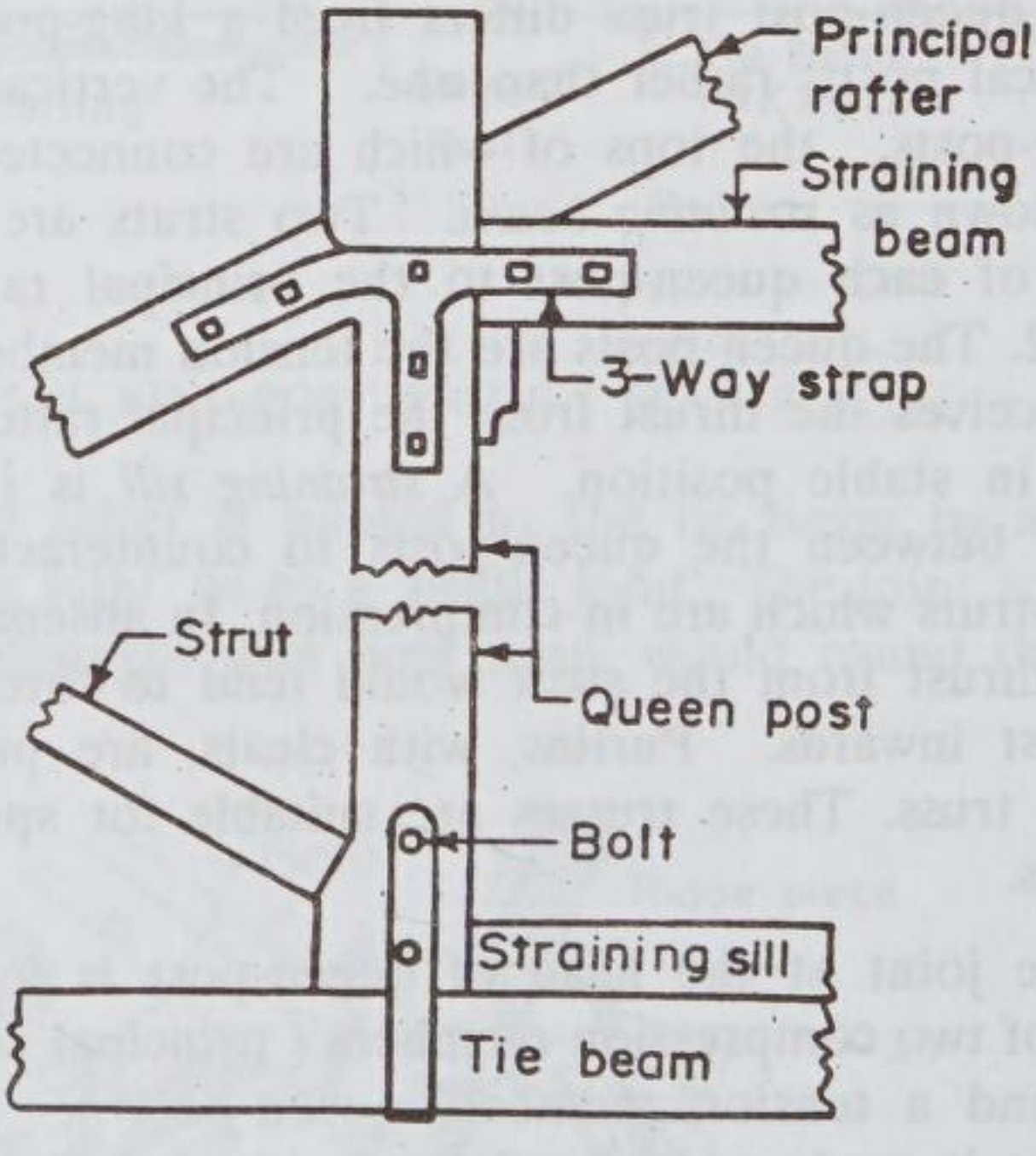
2. Queen-post truss

A queen-post truss differs from a king-post truss in having two vertical posts, rather than one. The vertical posts are known as queen-posts, the tops of which are connected by a horizontal piece, known as *straining beam.* Two struts are provided to join the feet of each queen-post to the principal rafter, as shown in Fig. 15.12. The queen-posts are the tension members. The straining beams receives the thrust from the principal rafters, and keeps the junction in stable position. A *straining sill* is introduced on the tie beam between the queen-posts to counteract the thrust from inclined struts which are in compression. In absence of the straining sill, the thrust from the strut would tend to force the foot of the queen-post inwards. Purlins, with cleats, are provided as in the king-post truss. These trusses are suitable for spans between 8 to 12 metres.

The joint at the head of queen-post is formed due to the junction of two compression members (principal rafter and straining beam) and a tension member(queen-post). The head of the queen-post is made wider, and the head of the principal rafter and the end of straining beam are tenoned into it. The joint is further strengthened by fixing a 3-way strap of wrought-iron or steel on each face as shown in Fig. 15.12 (*b*). Similarly, the feet of queen-post is widened to receive the tenon of the inclined strut, forming a 'single abutment and tenon joint'. The queen-post then tenons into the tie beam. The joint is further strengthened by stirrup straps and bolts.



(a) Queen Post Truss



(b) Joint Details

FIG. 15.12. QUEEN-POST TRUSS.

3. Combination of king-post and queen-post trusses

Queen-post trusses are suitable for spans upto 12 metres. For greater spans, the queen-post truss can strengthened by one more upright member, called *princess-post* to each side. Fig. 15.13 (*a*) and (*b*) show the resulting combination of king-post and queen-post trusses, which are suitable upto 18 m span.

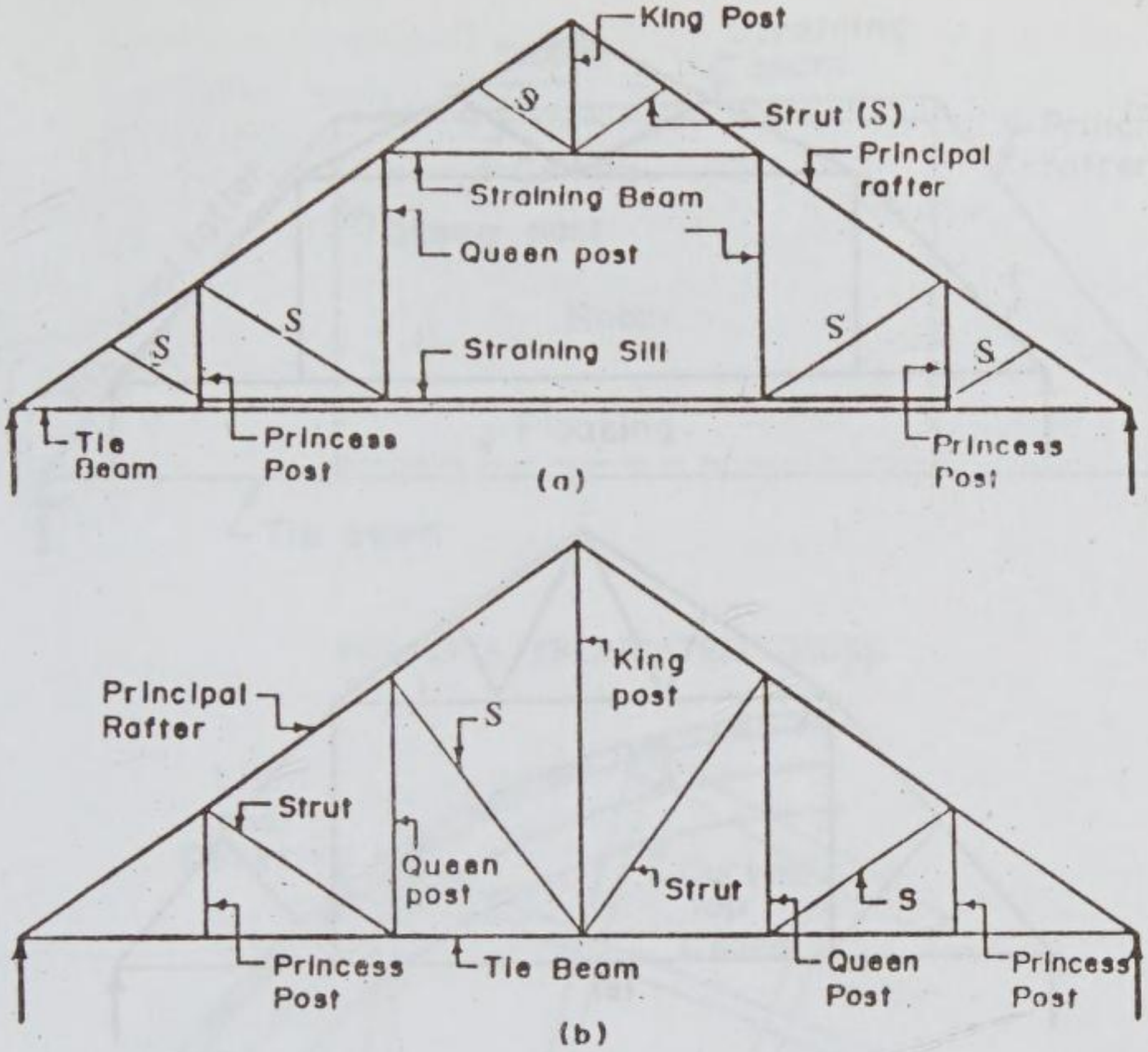


FIG. 15.13. COMBINATION OF KING-POST AND QUEEN-POST TRUSSES.

4. Mansard roof truss

This roof truss, named after its designer Francois Mansard, a French architect, is a combination of king-post and queen-post trusses. It is a two-storey truss, with upper portion consisting of king-post truss and the lower portion of queen-post truss. The entire truss has two pitches. The upper pitch (king-post truss) varies from 30° to 40° while two lower pitch (queen-truss) varies from 60° to 70°. The use of this truss results in economy in space, since a room may be provided between the two queen-posts. However, it has become obsolete because of odd shape. Fig. 15.14 shows two alternative forms of Mansard truss. Fig. 15.15 shows the details of the truss.

5. Truncated truss

A truncated truss is similar to Mansard truss, except that its top is formed flat, with a gentle slope to one side. This type of truss is used when it is required to provide a room in the roof, between the two queen-posts of the truss, as shown in Fig. 15.16.

6. Bel-fast roof truss (Bow string truss)

This truss, in the form of a bow, consists of thin sections of timber, with its top chord curved. If the roof covering is light, this roof truss can be used upto 30 m span. The roof truss is also known as *latticed roof truss.*

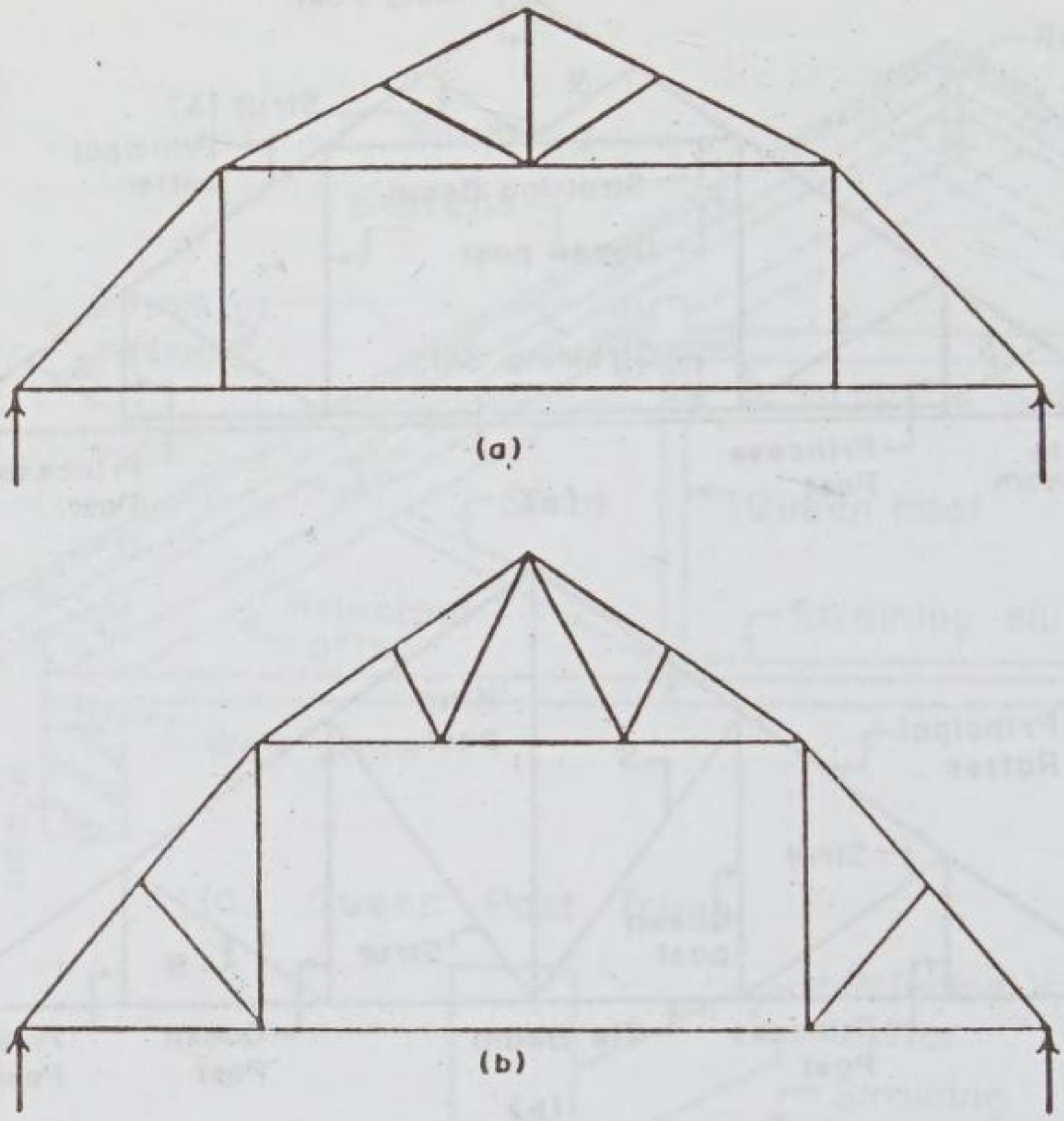


FIG. 15.14. ALTERNATIVE FORMS OF MANSARD TRUSSES.

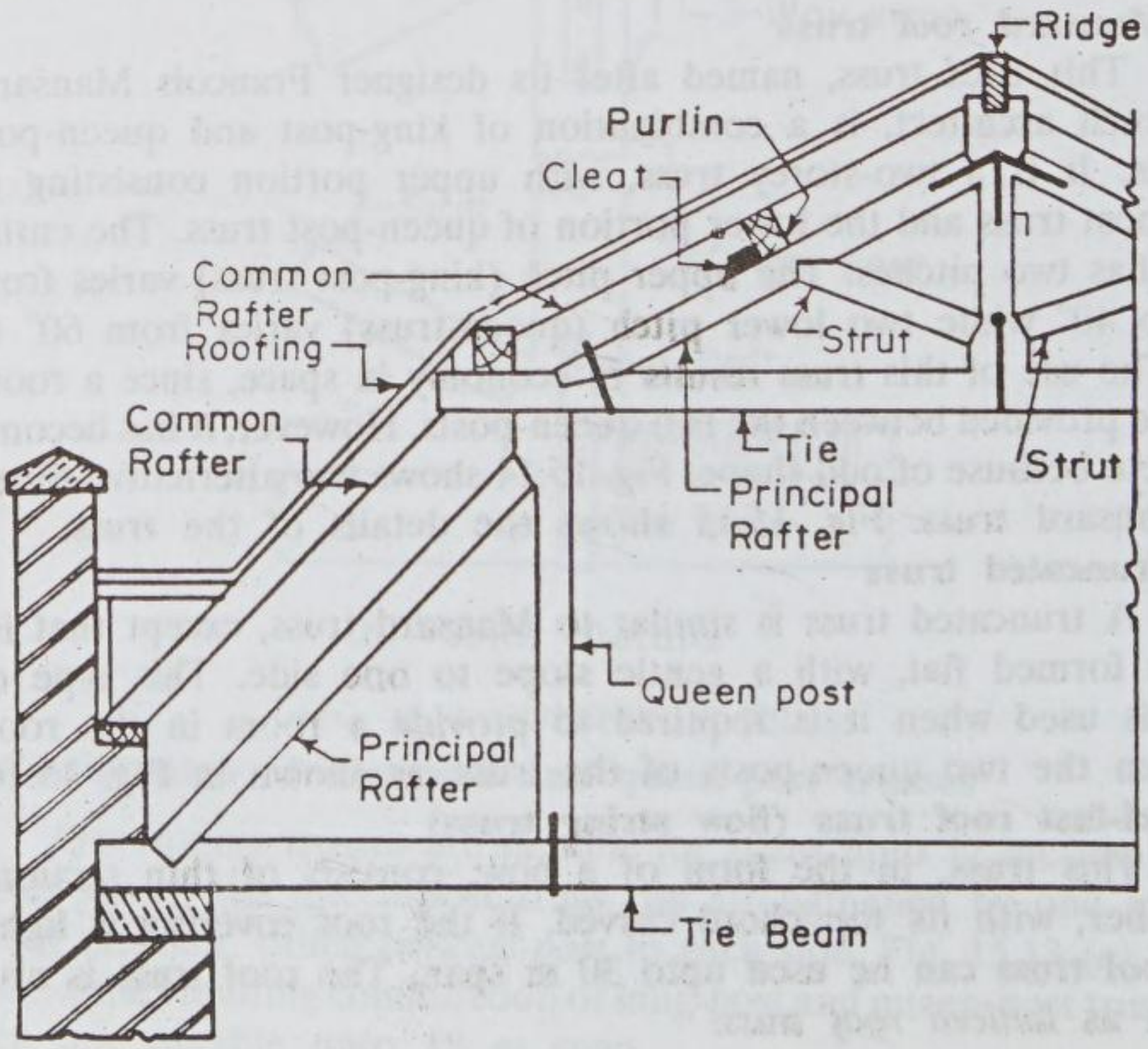


FIG. 15.15. DETAILS OF MANSARD TRUSS.

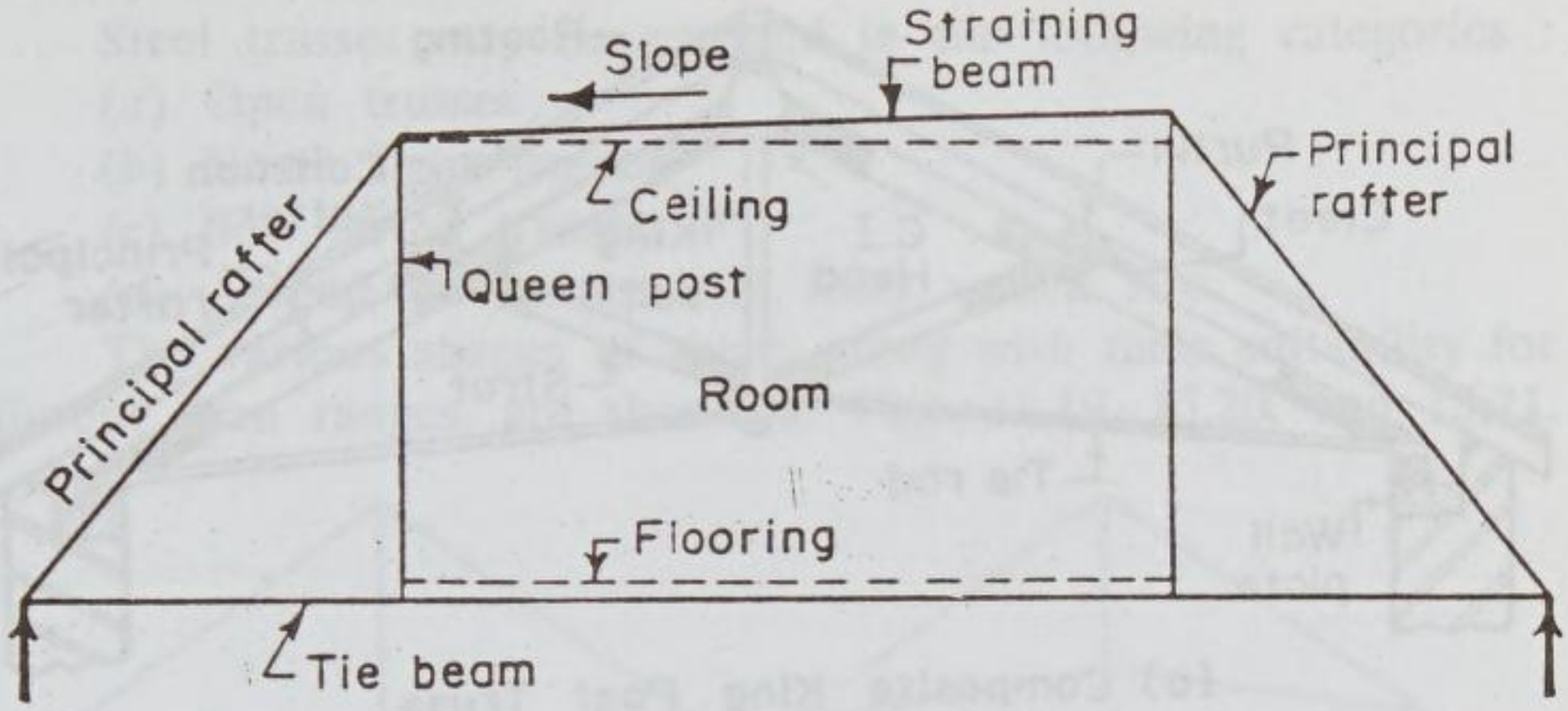


FIG. 15.16. TRUNCATED TRUSS.

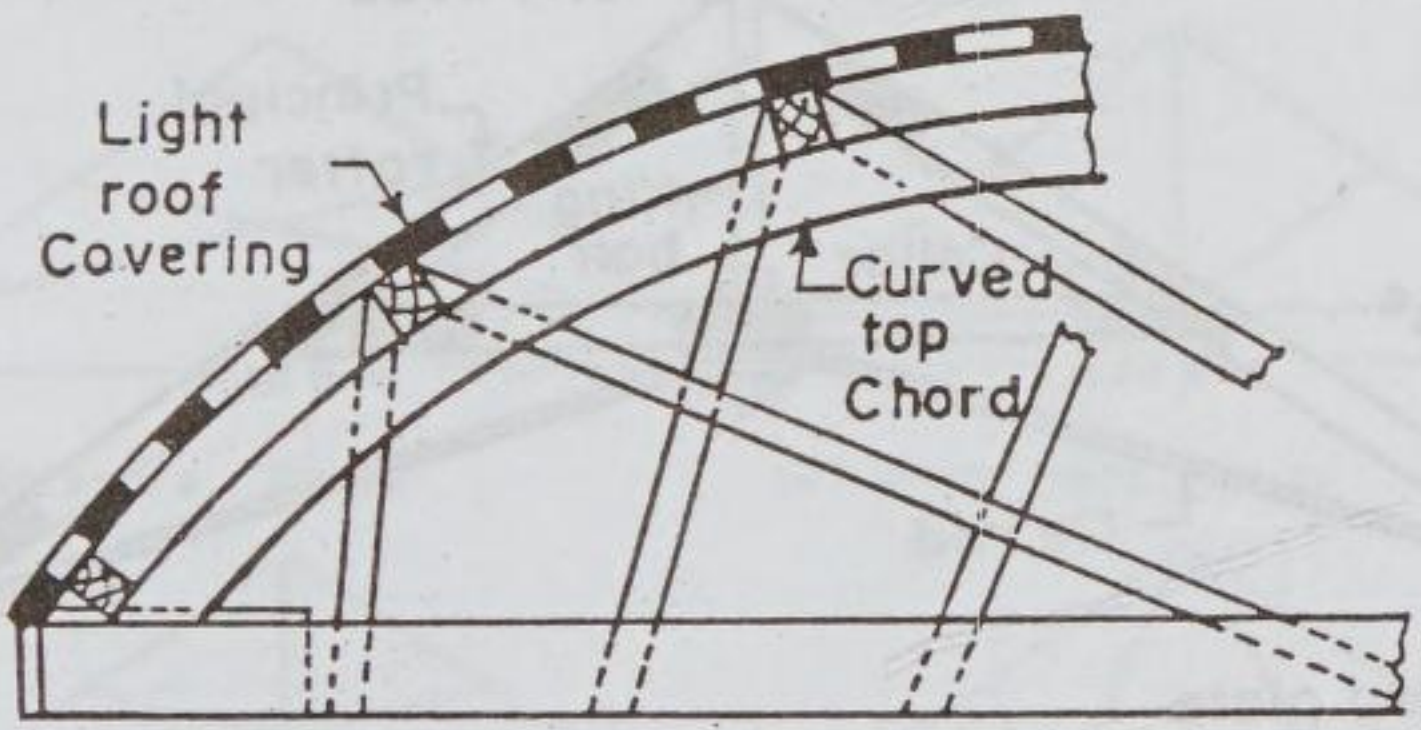


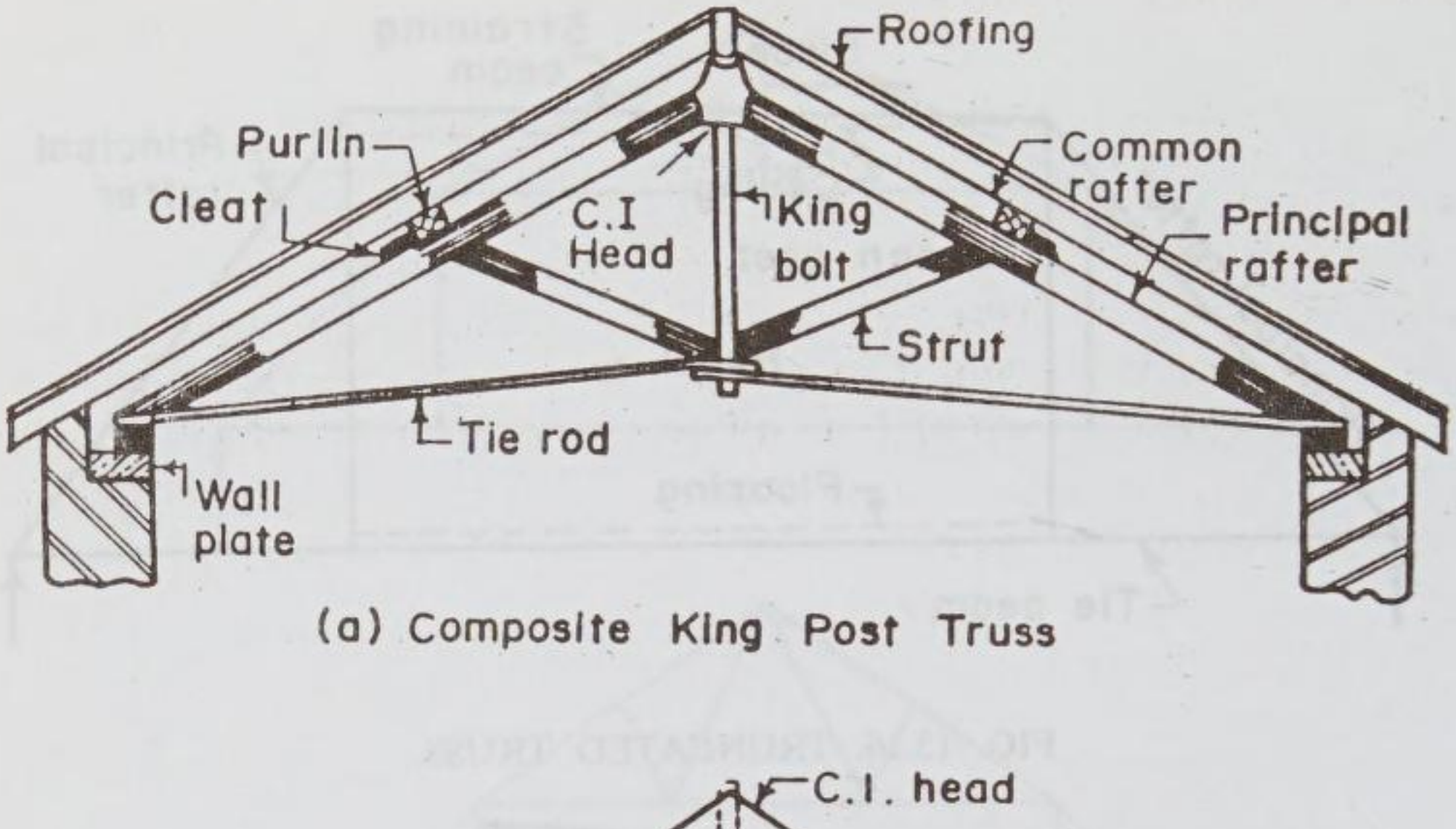
FIG. 15.17. BET-FAST TRUSS.

7. Composite roof trusses

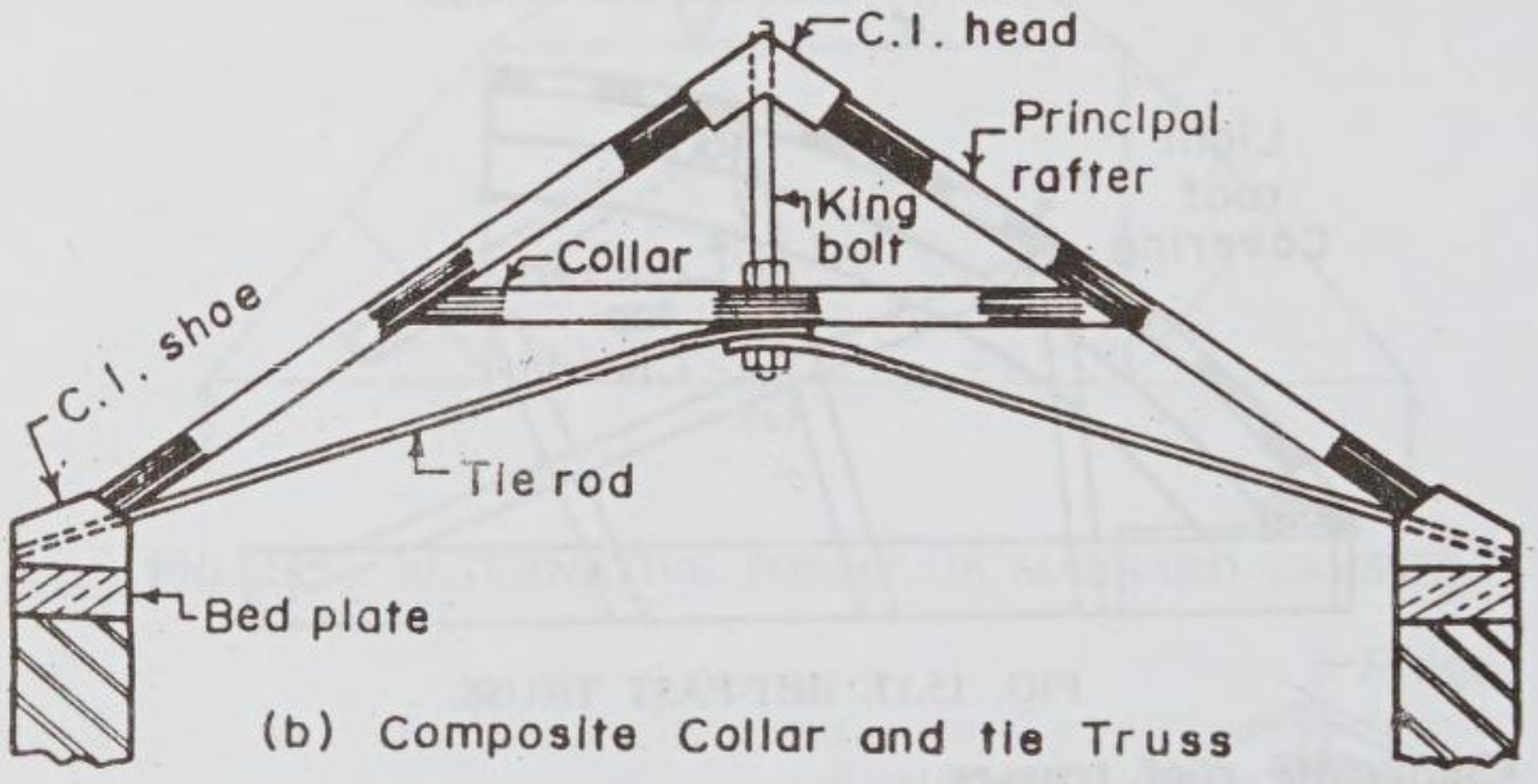
Roof trusses made of two materials, such as timber and steel, are known as *composite* roof trusses. In a composite truss, the tension members are made of steel, while compression members are made of timber. If tension members are made of timber, their section becomes very heavy because of reduction of section at the joints. Special fittings are required at the junction of steel and timber members. The joints in composite trusses should be such that cast or forged fittings can be easily used. Fig. 15.18 shows some common types of composite roof trusses, using fittings such as C.I. head, C.I. shoe, steel angle bolts and straps etc.

15.8. STEEL ROOF TRUSSES

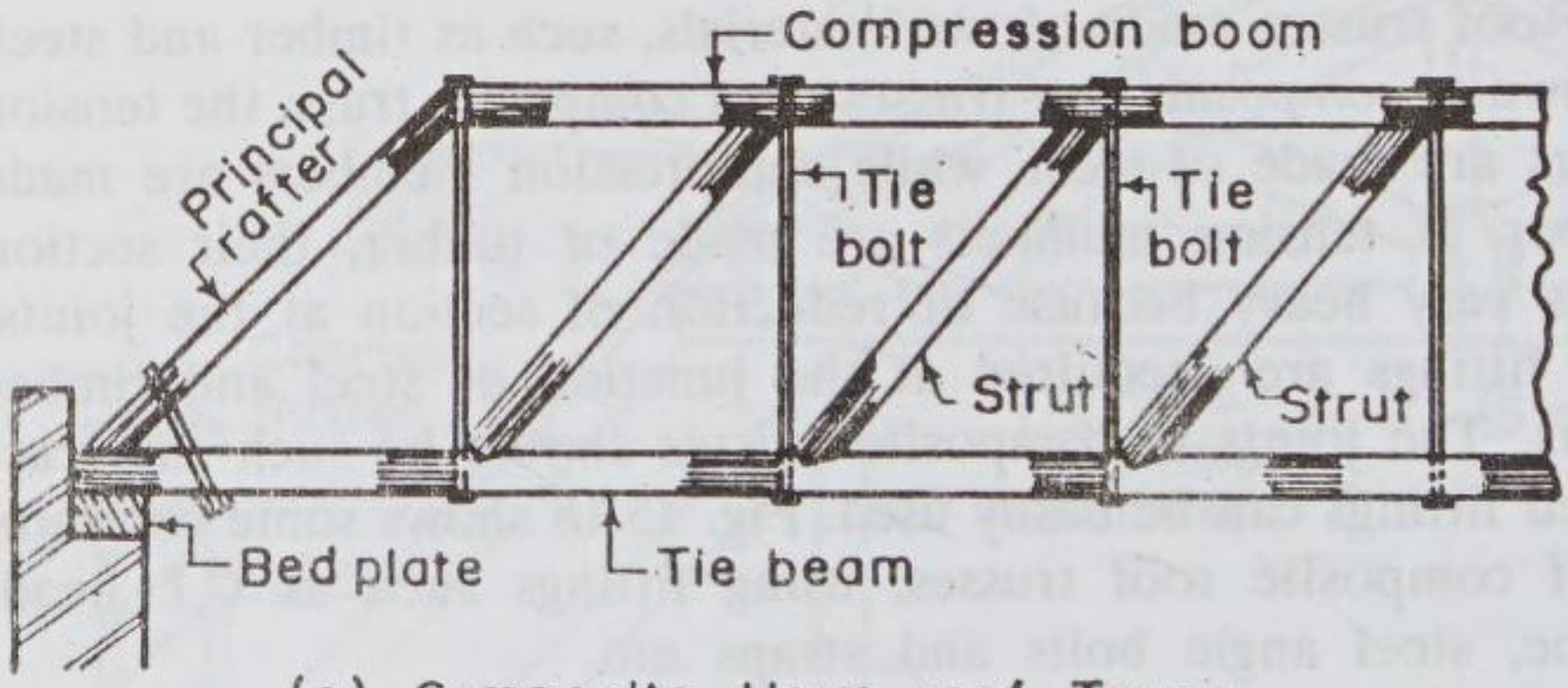
When the span exceeds 10 m, timber trusses become heavy and uneconomical. Steel trusses are more economical for larger spans. However, steel trusses are more commonly used these days, for all spans–small or large, since they are : (*i*) more economical, (*ii*) easy to construct or fabricate, (*iii*) fire-proof, (*iv*) more rigid, and (*v*) permanent. Steel trusses are fabricated from rolled steel structural



(a) Composite King Post Truss



(b) Composite Collar and tie Truss



(c) Composite Howe roof Truss

FIG. 15.18. COMPOSITE ROOF TRUSS.

members such as channels, angles, T-sections and plates. Most of the roof trusses are fabricated from angle-sections because they can resist effectively both tension as well as compression, and their jointing is easy. In India, where timber has become very costly (except in hilly regions), steel trusses have practically superseded timber trusses.

Steel trusses may be grouped in the following categories :

(*a*) Open trusses

(*b*) North light trusses

(*c*) Bow string trusses

(*d*) Arched rib trusses and solid arched ribs.

The various shapes of these, along with their suitability for different span ranges, are shown in Figs. 15.19, 15.20, and 15.21.

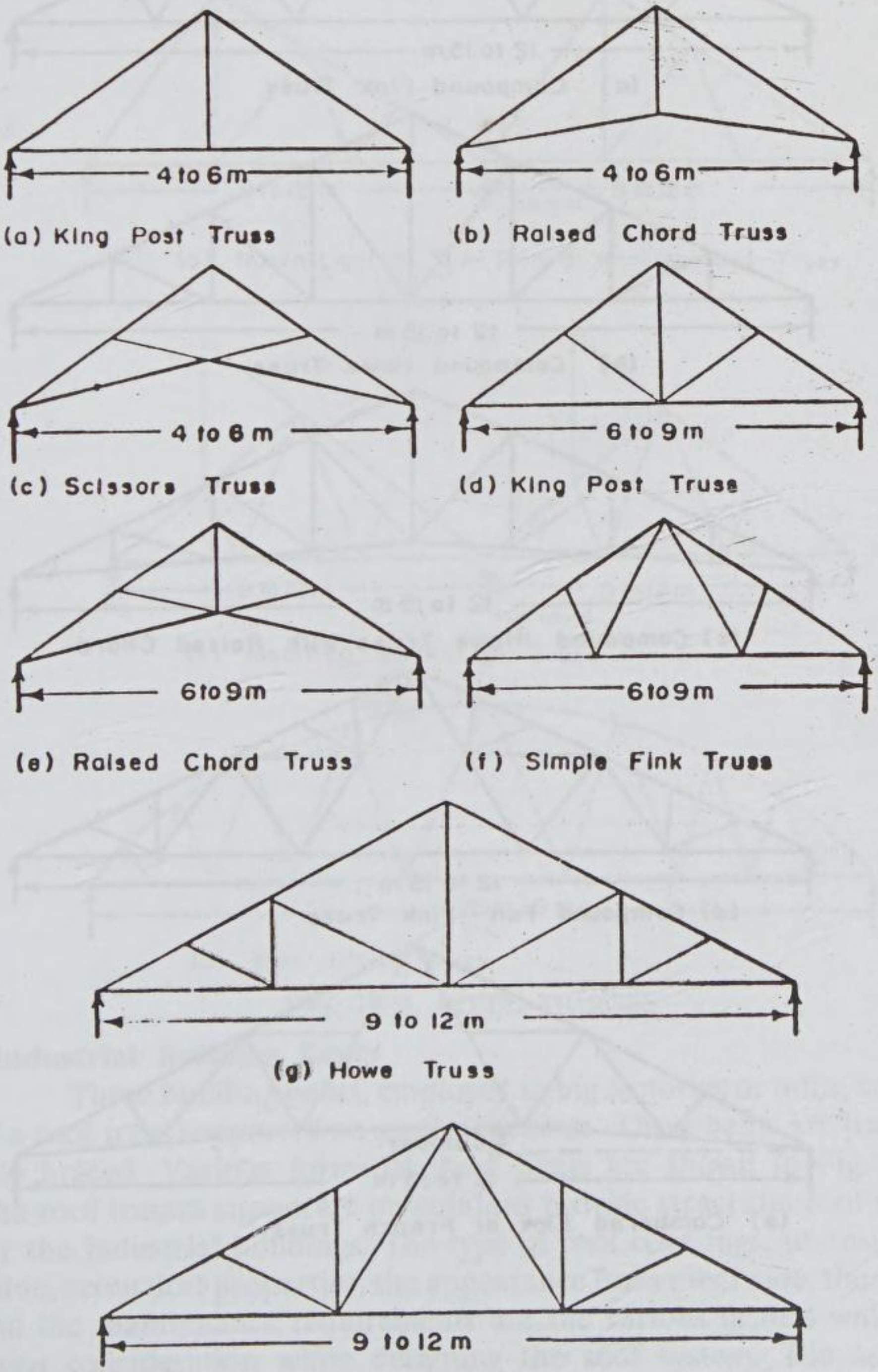
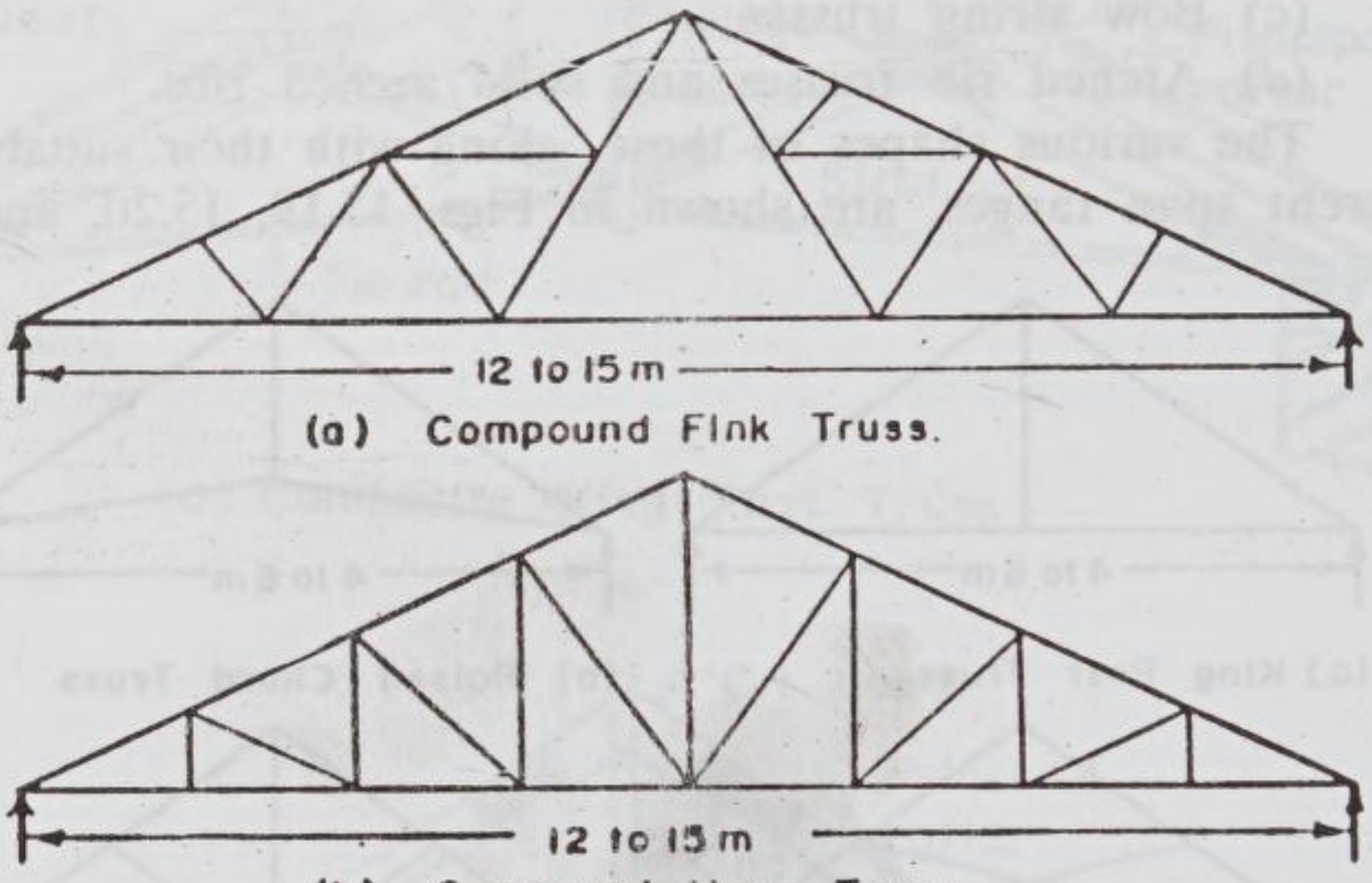
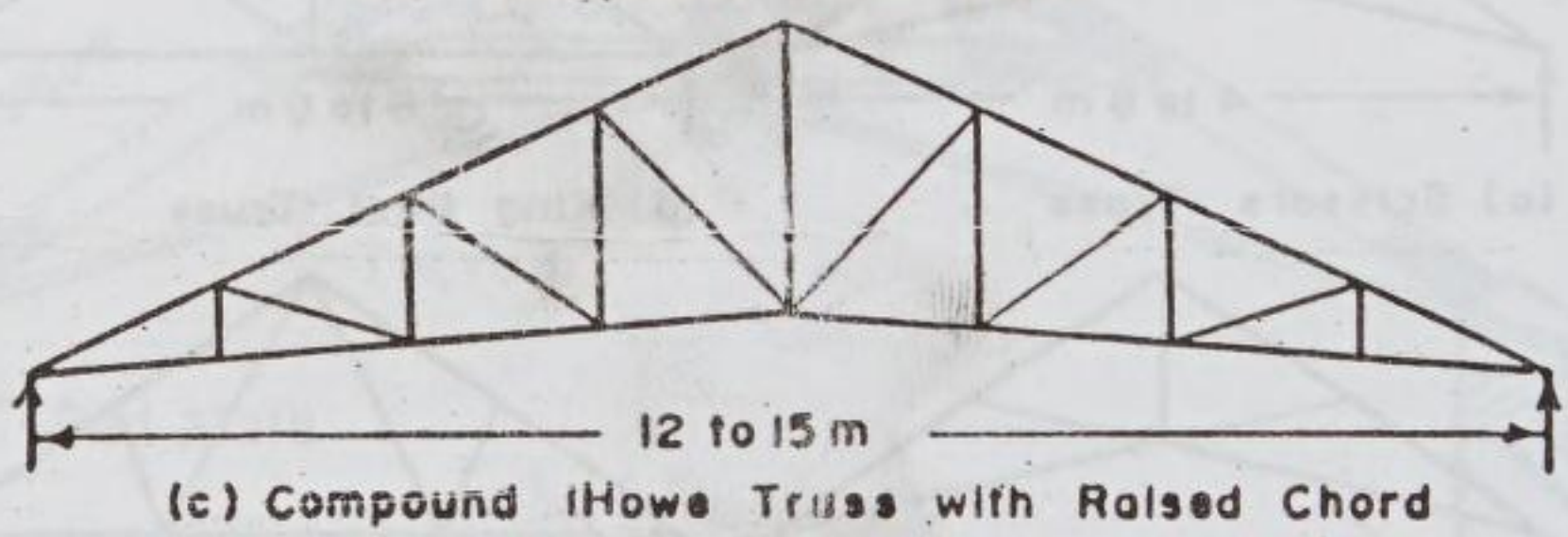


FIG. 15.19. STEEL TRUSSES.

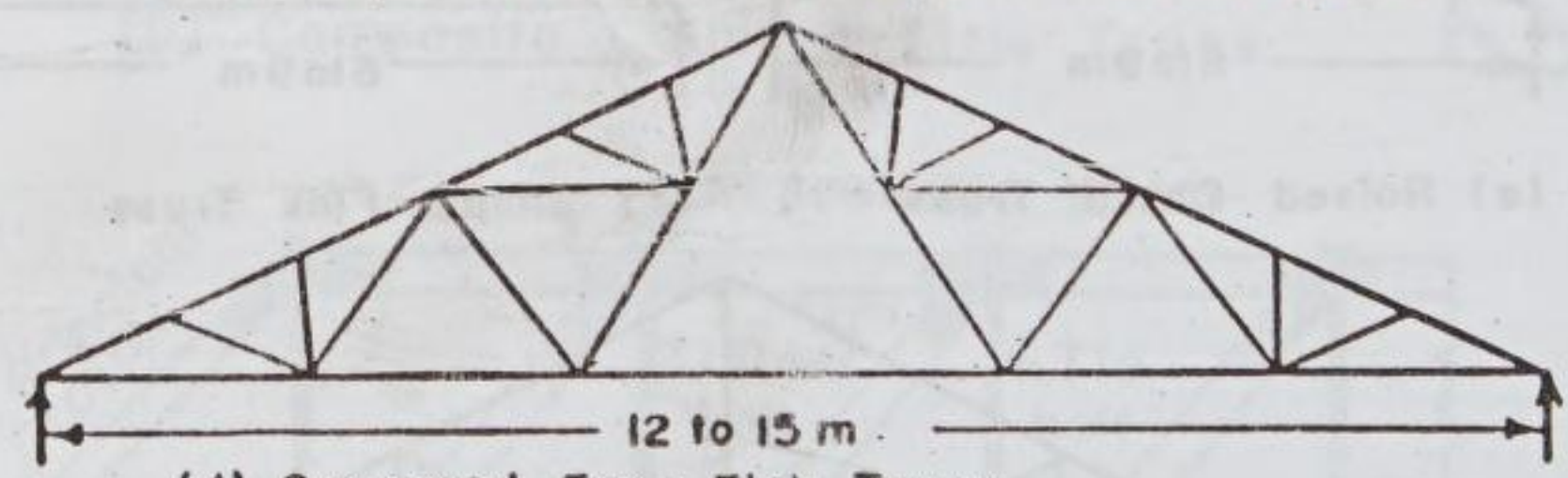


(a) Compound Fink Truss.

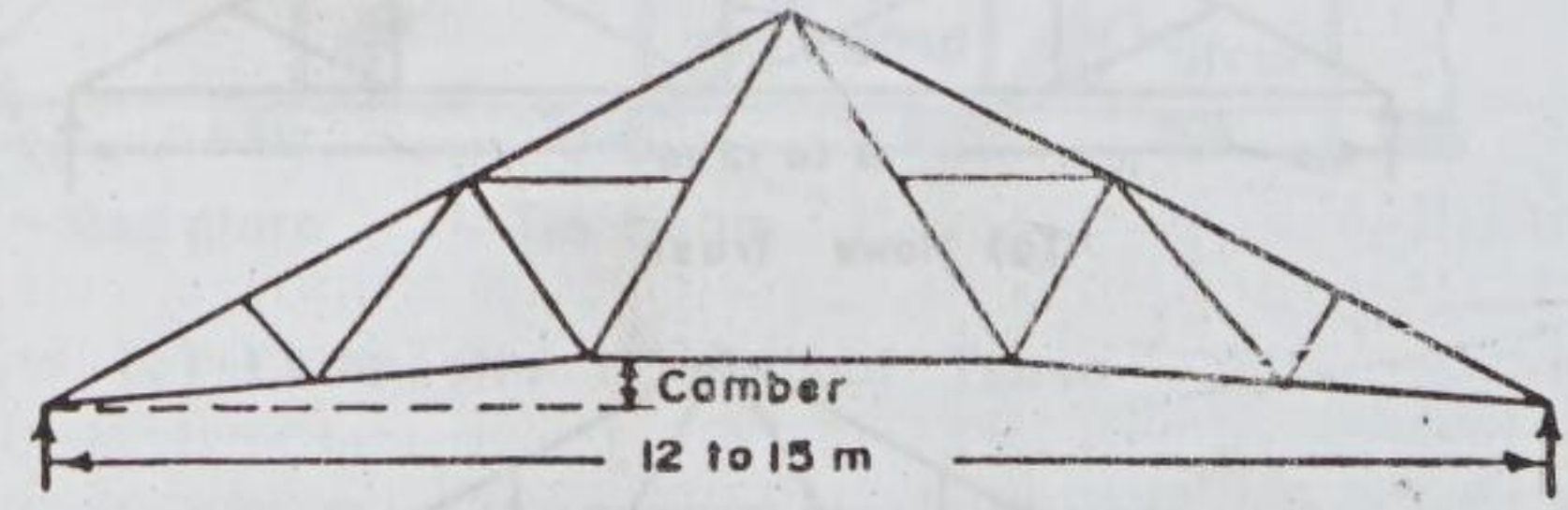
(b) Compound Howe Truss



(c) Compound iHowe Truss with Raised Chord

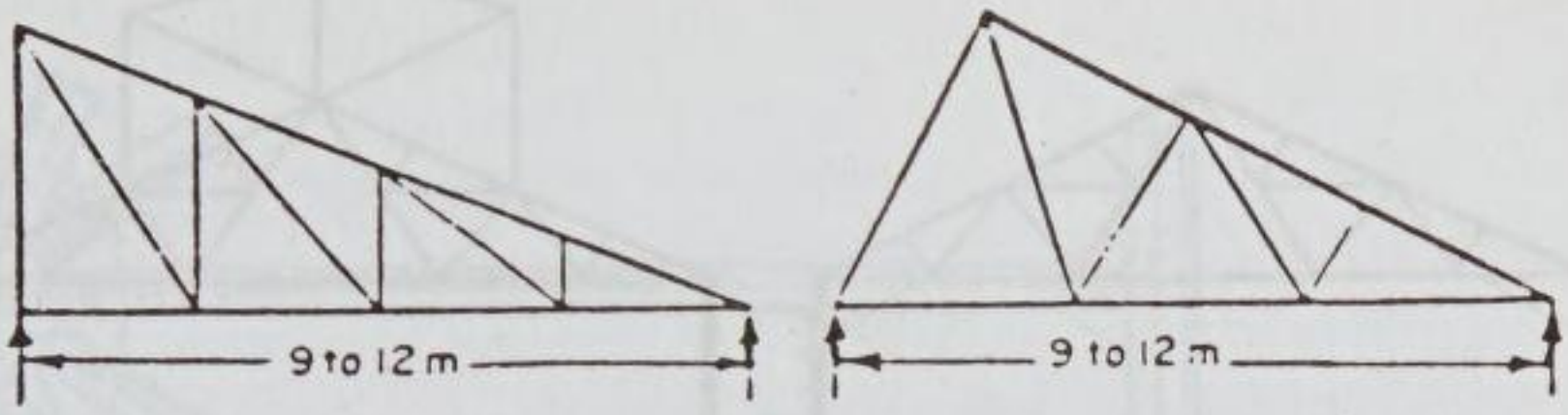


(d) Compound Fan - Fink Truss

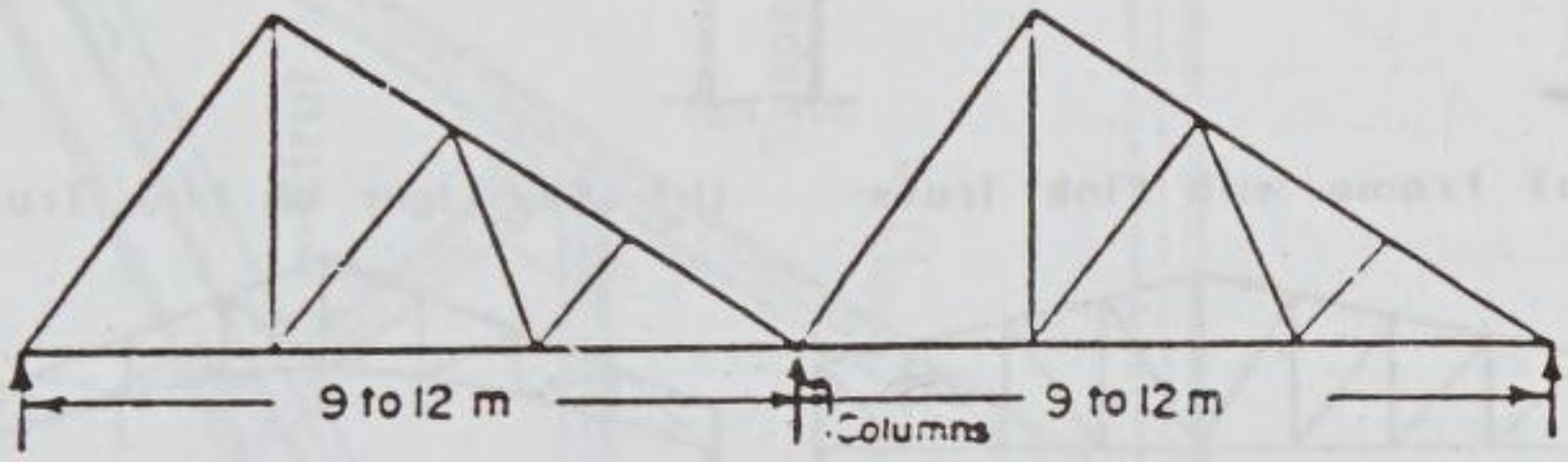


(e) Cambered Fink or French Truss

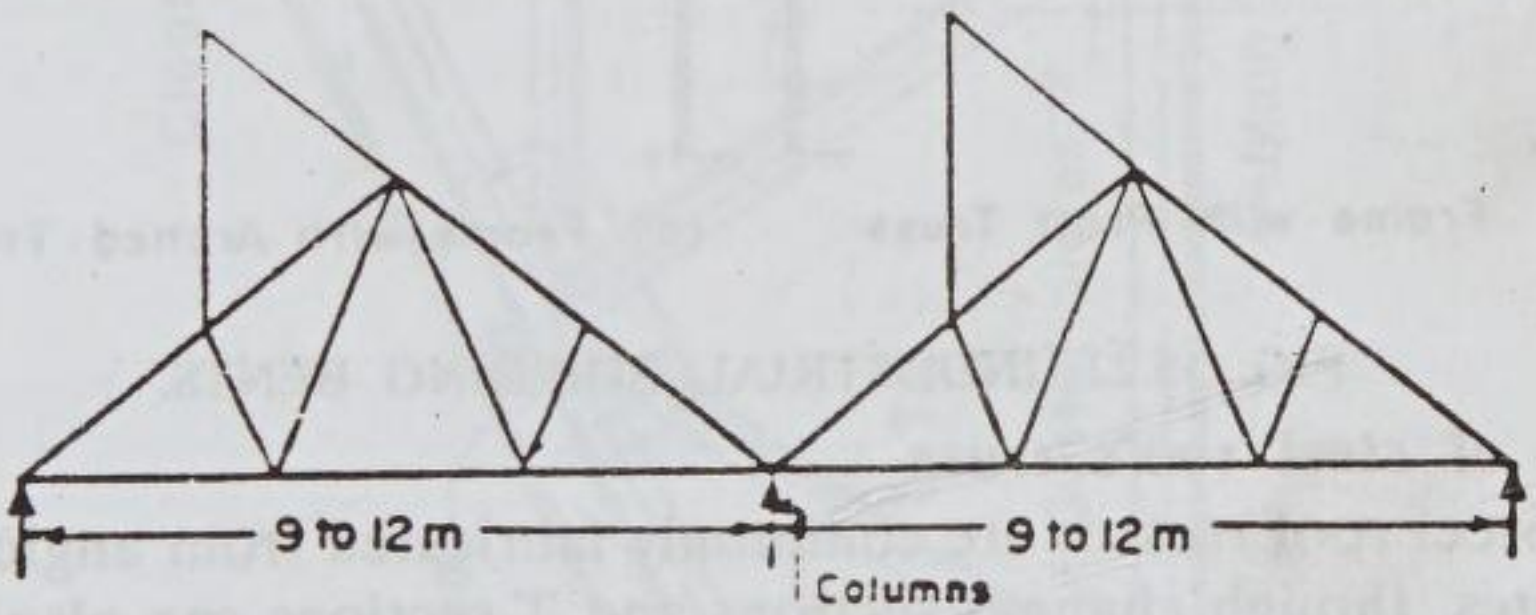
FIG. 15.20. STEEL TRUSSES.



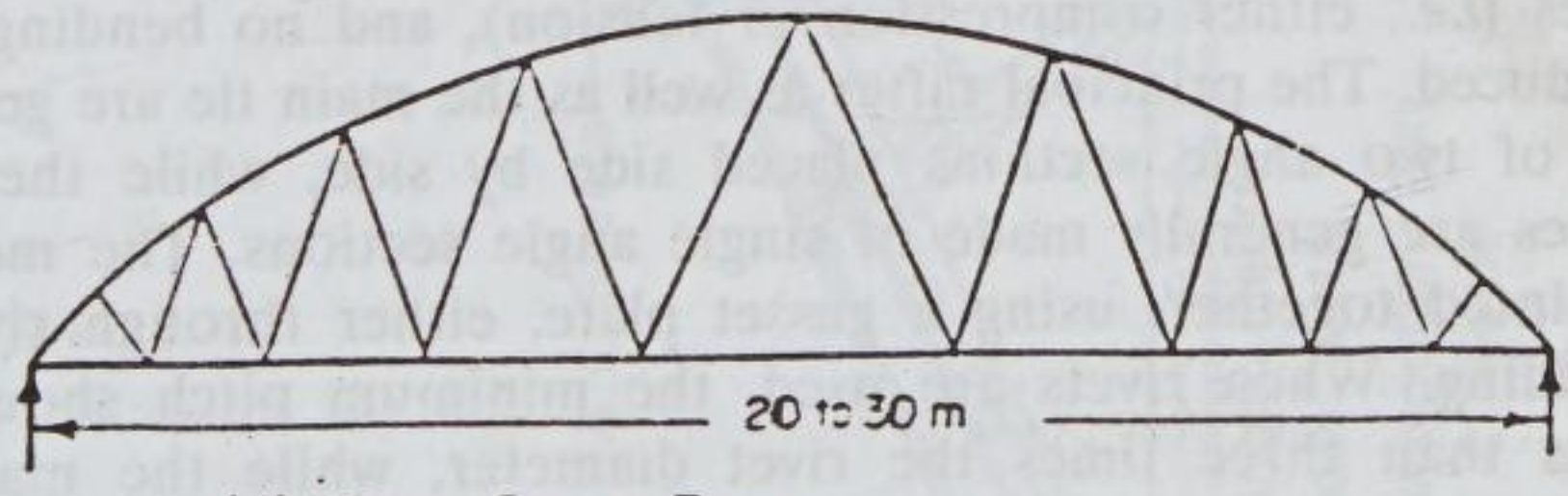
(a) North Light Trusses



(b) North Light or Saw-Tooth or Weaving shed Truss



(c) Modified North Light Truss



(d) Bow String Truss

FIG. 15.21. STEEL TRUSSES.

Industrial Building Bents

These building bents, employed in big factories or mills, consists of a roof truss supported on steel stanchions. These bents are transversely braced. Various forms of these bents are shown in Fig. 15.22. The roof trusses supported on columns provide structural roof system for the industrial buildings. The type of roof coverings, its insulating value, acoustical properties, the appearance from inner side, the weight and the maintenance requirements are the various factors which are given consideration while designing the roof system. The asbestos corrugated and trafford cement sheets, and the galvanised corrugated sheets are commonly used as the roof covering materials.

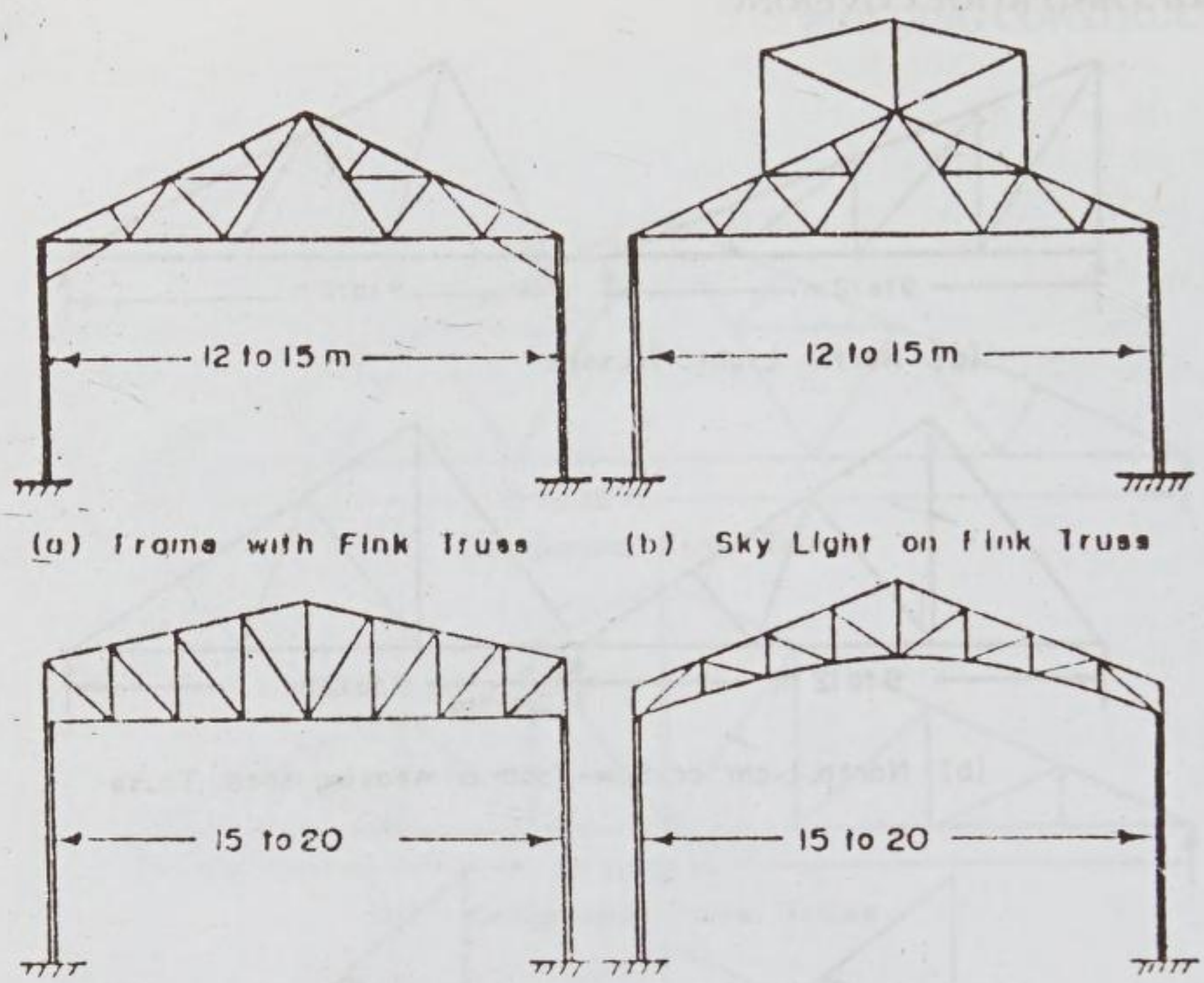

12 to 15 m
12 to 15 m
(a) Frame with Fink Truss
(b) Sky Light on Fink Truss
15 to 20
15 to 20
(c) Frame wlih Pratt Truss
(d) Frame with Arched Truss

FIG. 15.22. INDUSTRIAL BUILDING BENTS.

Details of steel roofs truss

Steel roof trusses are commonly fabricated from angle sections and plates, though channel sections and T-sections can also be used. The roof truss is so designed that the members carry only direct stresses (*i.e.*, either compression or tension), and no bending stress are induced. The principal rafter as well as the main tie are generally made of two angle sections placed side by side, while the struts and ties are generally made of single angle sections. The members are jointed together, using a gusset plate, either through rivets or by welding. When rivets are used, the minimum pitch should not be less than three times the rivet diameter, while the maximum pitch is limited to 15 cm for compression members and 20 cm for tension members. Generally, 15 mm diameter rivets are used for small spans and 20 mm rivets are used for large spans. At least two rivets should be used at each joint. Gusset plate should not be less than 6 mm, though its thickness is designed on the basis of forces carried by members to be jointed. At the foot of the truss, short angles are fitted on both the sides of the gusset plate, which are connected to the bearing plate. The bearing plate is jointed to concrete bed through rag bolts. At the apex, suitable ridge section is fitted.

Steel trusses have the following advantages over timber trusses :

1. The sections comprising of a steel truss are readily available

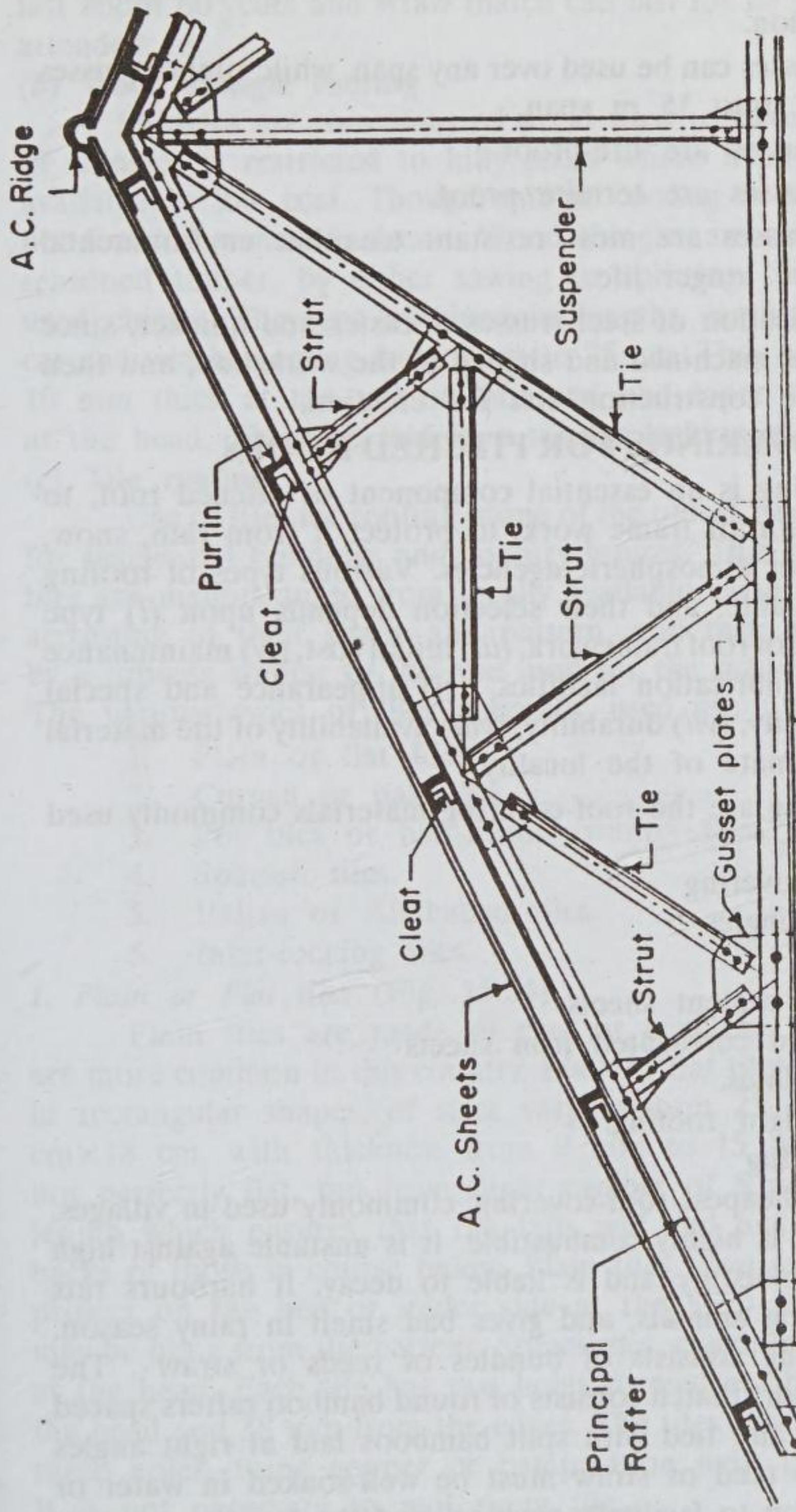


FIG. 15.23. DETAILS OF STEEL ROOF TRUSS.

in the required dimensions, resulting in minimum wastage of material.

2. Steel trusses are light in weight, and can be fabricated in any shape depending upon structural and architectural requirements.

3. Steel trusses are stronger and more rigid in comparison

to timber trusses. The members are equally strong in tension as well as compression.

4. Steel trusses can be used over any span, while timber trusses are suitable only upto 15 m span.

5. Steel trusses are fire-proof.

6. Steel trusses are termite proof.

7. Steel trusses are most resistant to other environmental agencies, and have longer life.

8. The fabrication of steel trusses is easier and quicker, since the sections can be machined and shaped in the workshop, and then transported to the construction site for erection.

15.9. ROOF COVERINGS FOR PITCHED ROOFS

Roof covering is an essential component of pitched roof, to be placed over the roof frame work, to protect it from rain, snow, sun, wind and other atmospheric agencies. Various types of roofing materials are available, and their selection depends upon (*i*) type of building, (*ii*) type of roof framework, (*iii*) initial cost, (*iv*) maintenance requirements, (*v*) fabrication facilities, (*vi*) appearance and special features of the locality, (*vii*) durability, (*viii*) availability of the material itself, and (*ix*) climate of the locality.

The following are the roof-covering materials commonly used for pitched roofs :

1. Thatch covering
2. Wood shingles
3. Tiles
4. Asbestos cement sheets
5. Galvanised corrugated iron sheets
6. Eternit slates.
7. Light weight roofing.

(*a*) Thatch covering

This is the cheapest roof-covering, commonly used in villages. It is very light, but is highly combustible. It is unstable against high winds. It absorbs moisture and is liable to decay. It harbours rats, and other burrowing animals, and gives bad smell in rainy season. Thatch roof-covering consists of bundles of reeds or straw . The frame work to support thatch consists of round bamboo rafters spaced 20 to 30 cm apart and tied with split bamboos laid at right angles to the rafters. The reed or straw must be well-soaked in water or fire-resisting solution to facilitate packing, and the bundles are laid with their butt ends pointing towards the eaves. The thatch is tightly secured to the frame work with the help of ropes or twines dipped in tar. In order to drain the roof effectively, a minimum slope of 45° is kept. The thickness of thatch covering should at least be 15 cm ; normal thickness varies from 20 to 30 cm according to

its quality and pitch of roof. It is claimed that reed thatch can last about 60 years and straw thatch can last for 20 years, if properly attended to.

(*b*) Wood shingle roofing

Shingles are thin slabs of wood used to cover roofs. The use of shingles is restricted to hilly areas where local timber is easily available at low cost. Though shingle roofing is light weight, it is not fire and termite resistant. Wood shingles are obtained from well seasoned timber, by either sawing or splitting. Sawn shingles are used chiefly. They are obtained in lengths varying from 30 to 40 cm and widths varying from 6 cm to 25 cm. They are approximately 10 mm thick at the tail or butt end and taper to 3 mm or less at the head. They are laid in a similar fashion as tiles and slates.

(*c*) Tile roofing

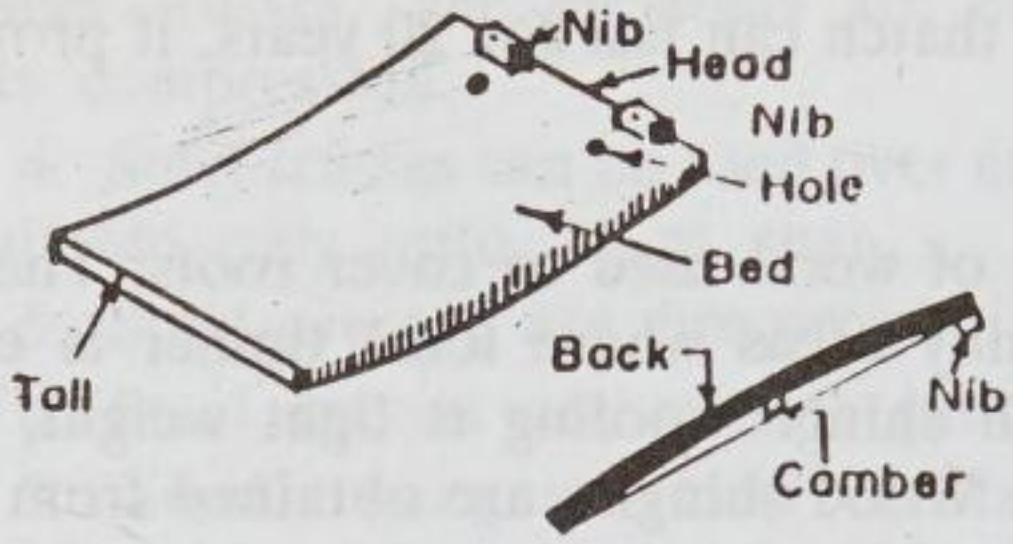
Use of tiles for roofing is one of the oldest, and is still preferred for residential buildings and country houses. This is because *country tiles* are manufactured from locally available earth. Tiles are named according to their shape and pattern, and they are manufactured by a process similar to the one used for the manufacture of bricks. The various types of tiles generally used are :

1. Plain or flat files.
2. Curved or pan-tiles.
3. Pot tiles or half-round country tiles.
4. Spanish tiles.
5. Italian or Allahabad tiles.
6. Inter-locking tiles.

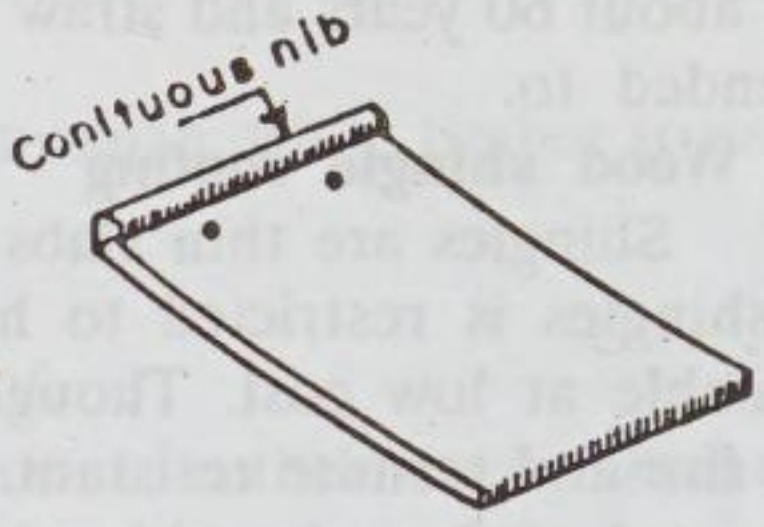
1. Plain or Flat tiles (Fig. 15.24)

Plain tiles are made of clay or concrete, though clay tiles are more common in this country. Plain or flat tiles are manufactured in rectangular shapes, of sizes varying form 25 cm×15 cm to 28 cm×18 cm, with thickness from 9 mm to 15 mm. The tiles are not perfectly flat, but have slight camber of 5 to 10 mm in their length which ensures that the tails will bed and not ride on the backs of those in course below. Plain tiles have *stubs* or *nibs* which project on the bed or under side at the head in order that they may be hung from the battens. Sometimes, *continuous nib* is provided at the head. Each tile has two holes formed at about 25 mm from the head and 38 mm from the edges. The tiles can be nailed through these holes, using copper or composition nails of 38 mm length. It is not necessary to nail every tile.

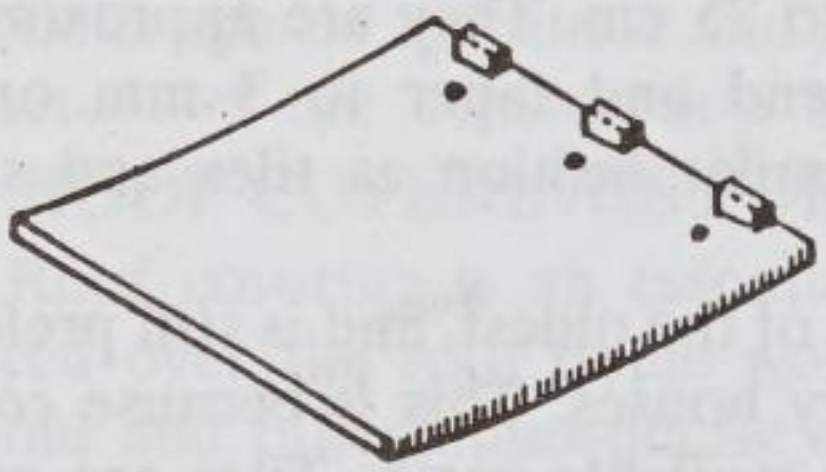
Before laying the tiles, common rafters are laid at 20 to 30 cm spacing. Battens or reepers are then fixed across the rafters at a spacing of 4 to 6 cm. The tiles are then laid over it with sufficient overlap on sides and edges. Plain tiles are laid in regular bond. For normal exposures, it is usual to nail every fourth or fifth course.



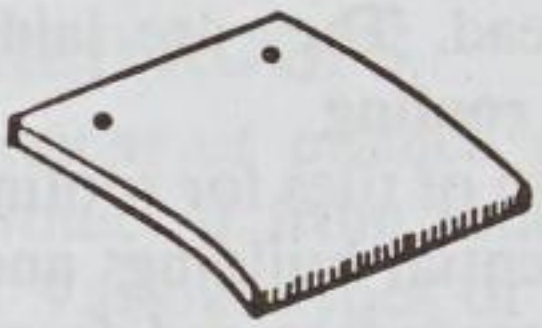
(a) Plain Tile with two Nibs



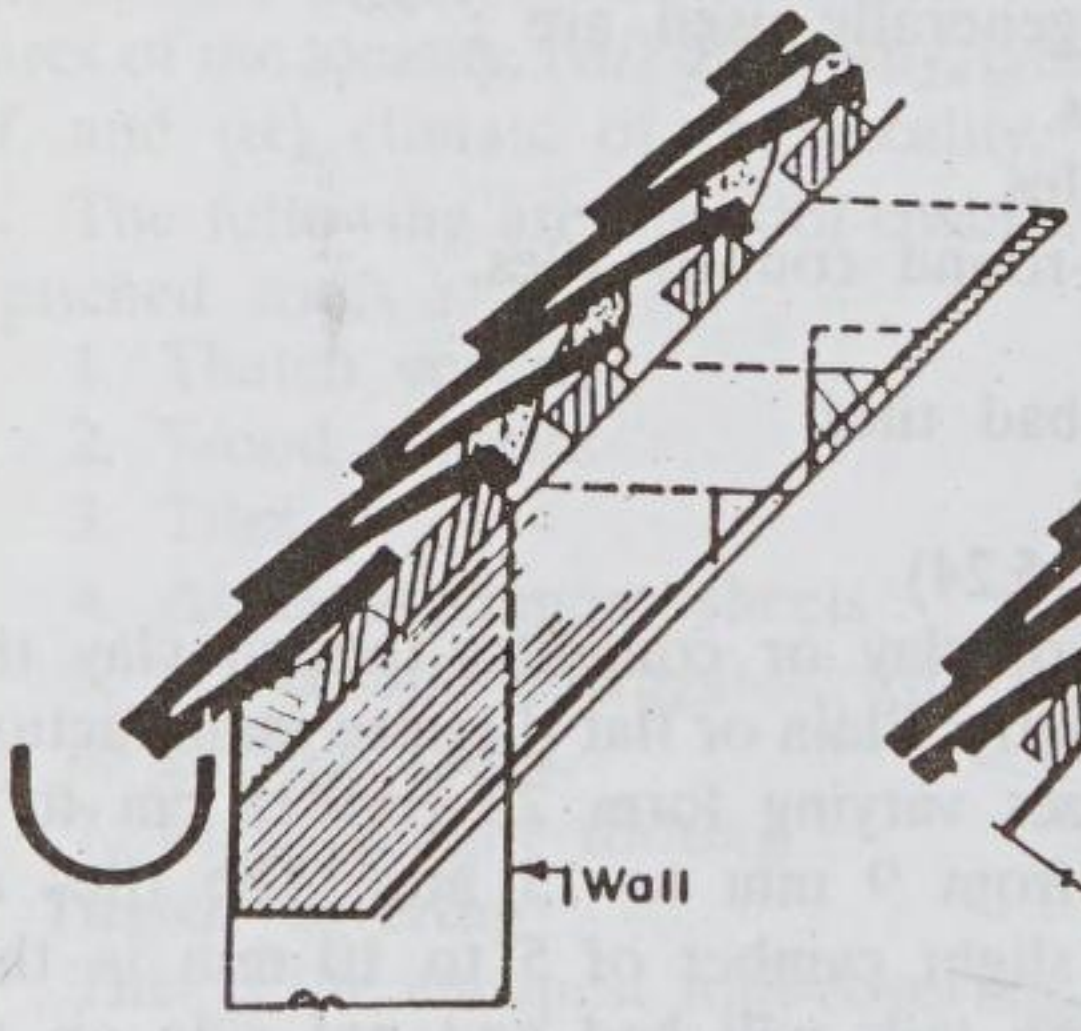
(a) Tile with Continuous Nib



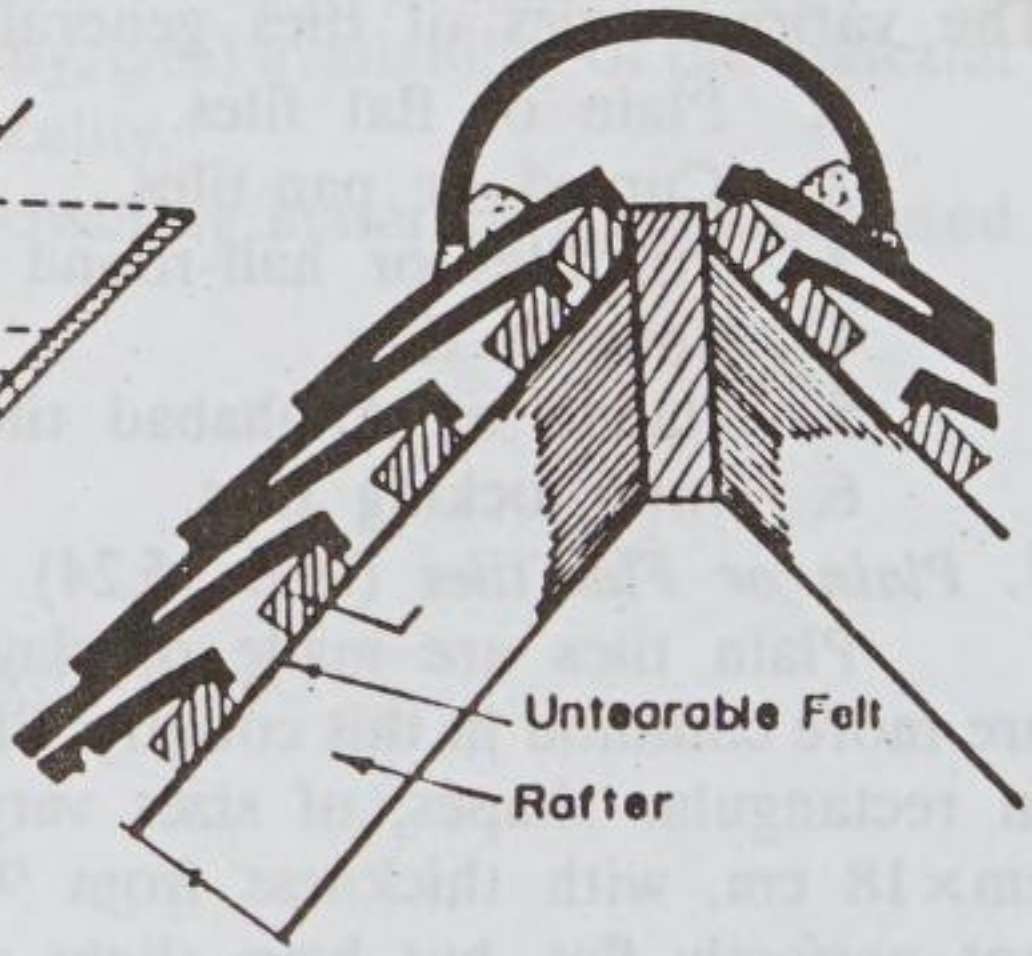
(c) Tile and a Half Tile



(d) Eaves Under-Tile



(e) Eaves Details



(f) Ridge Details

FIG. 15.24. ROOFING WITH PLAIN TILES.

However, in very exposed positions, specially if the roofs are steeply pitched, it may be necessary to nail every tile.

1. Hip and Valley details

Special tiles for the *under course* at eaves, top course at the ridges, and for hips and valley are used. At hip, special *granny bonnet hip tiles* (Fig. 15.25 *a*) are used. These hip tiles are bonded with the general plain tiles. Each hip tile is well-bedded with mortar on the back of the tile below and is secured with a long nail to the *hip rafter*.. Fig. 15.25 (*b*) shows the details of a valley formed with purpose made valley tiles.

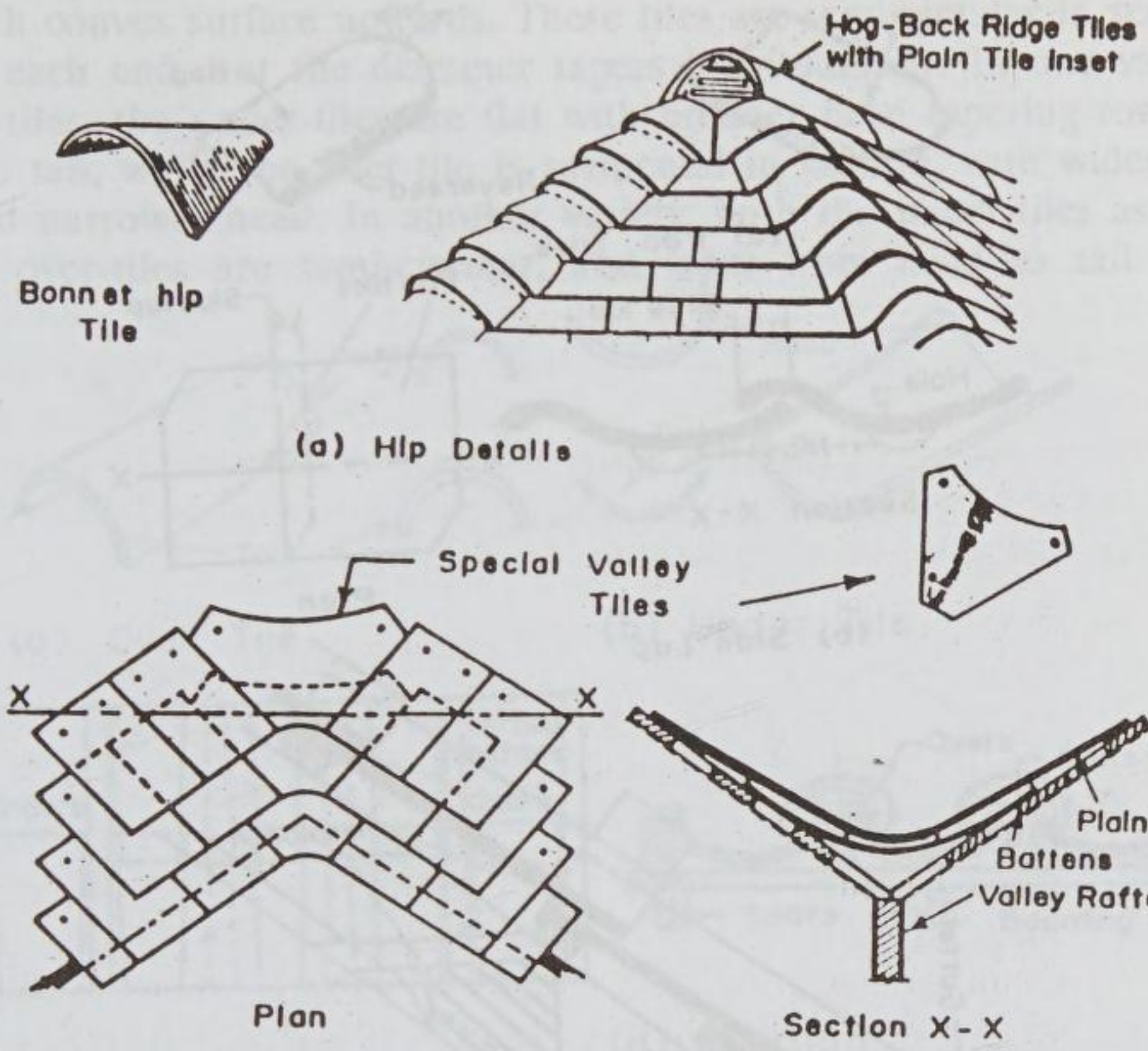


FIG. 15.25. HIP AND VALLEY DETAILLS.

2. Curved or pan-tiles (Fig. 15.26)

Pan-tiles are 33 to 36 cm long, 22.5 to 25 cm wide and 12 to 19 mm thick. They are flat longitudinally, but are curved transversely to a flat wave or S-curve. One nib is provided at the head on the underside of the trough of the wave, a nail hole is formed below the nib and two of the opposite diagonals corners are splayed or rounded. Pan-tiles are laid with *overlapping side joints* with two thicknesses only at the head joints and a single thickness at the unlapped portions. Pan-tiles are unbonded, having continuous side joints from eaves to the ridge. Thus, pan-tiles are *single lapped* in contrast to the plain tiles which are double lapped. Side lap in pan-tiles varies from 38 to 50 mm. The head or longitudinal lap varies from 7.5 to 10 cm, according to the pitch of the roof. Plain tiles are also nailed. As stated above, two diagonally opposite corners or *shoulders* are splayed off to the depth of the lap, to permit a reasonably close fit between the tiles. If this is not done, four thicknesses would occur at the corners, resulting in open joints due to tilting or over-riding of tiles. Fig. 15.26 shows the details of roofing with pan-tiles.

3. Half- round country tiles : Spanish tiles (Fig. 15.27)

Half-round country tiles are commonly used in villages. These

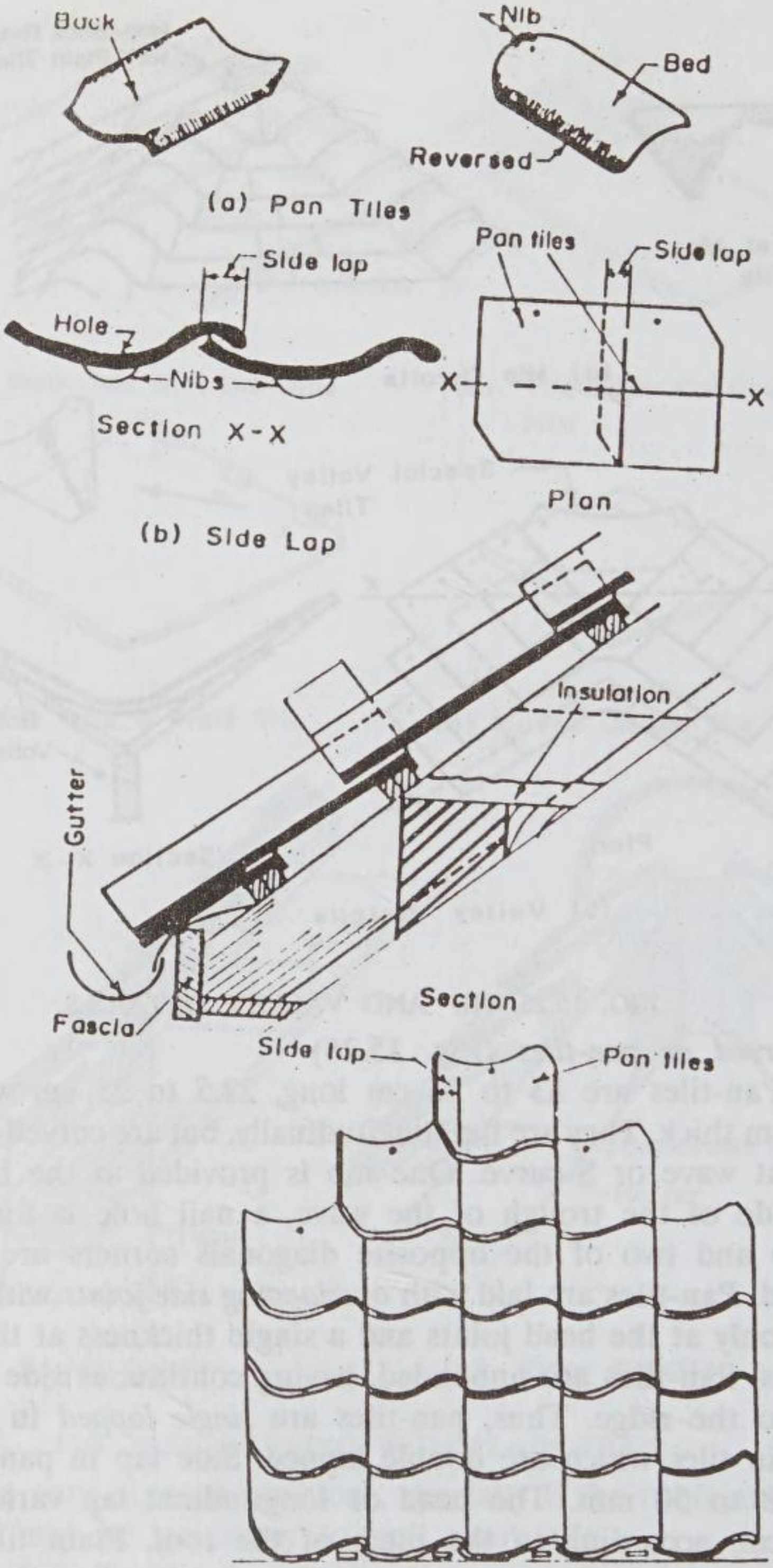


FIG. 15.26. ROOFING WITH PAN-TILES.

tiles are laid in pairs of *under-tiles* and *over-tiles*. The under-tiles are laid with concave surface upwards, while the over-tiles are laid

with convex surface upwards. These tiles are semi-circular in section at each end, but the diameter tapers longitudinally. In one variety of tiles, the under-tiles are flat with broader head tapering towards the tail, while the over-tile is segmental in section, with wider tail and narrower head. In another variety, both the under-tiles as well as over-tiles are semi-circular, and taper from head to tail.

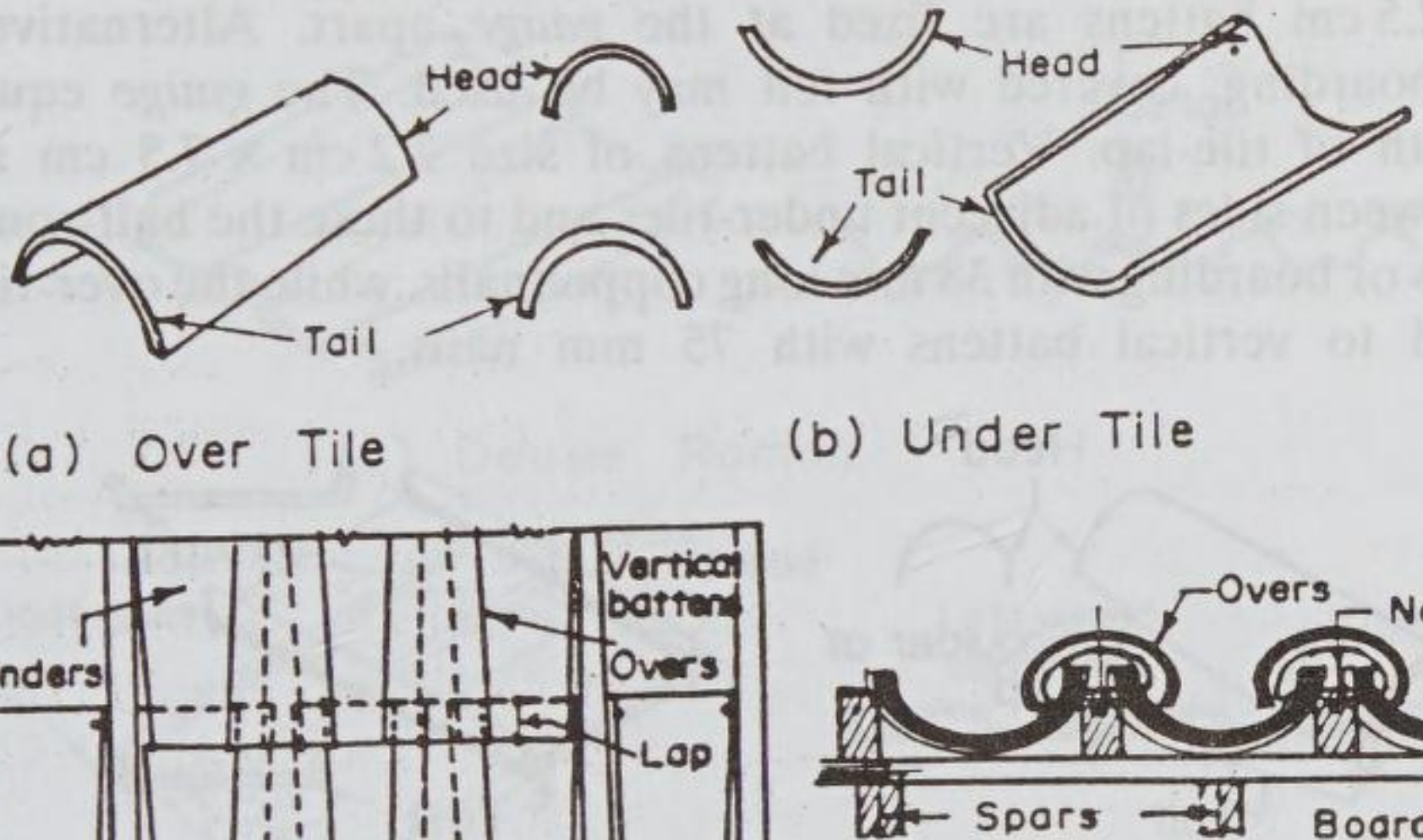


FIG. 15.27. ROOFING HALF-ROUND COUNTRY TILES OR SPANISH TILES.

The country tiles are similar to the spanish tiles. The overtiles taper down from tail to head while the under-tiles taper down from head to tail.

4. Italian or Allahabad tiles (Fig. 15.28)

These tiles are also used in pairs-flat broad bottom under-tile which alternate with convex curved over-tile. The *under-tile* is flat, tapered, with upturned edges or flanges at the sides. It measures 23 cm at the interior end (tail), 26 cm at the wide end (head) with a length of 37 cm and flange height of 4 cm. Italian tiles have the under-tile with flanges tapered, with a slight increase in depth towards the head. The over-tile is half-round in section and tapered in plan. The diameter tapers from 16 cm at tail to 12 cm

at the head. The tile may be slightly shouldered to allow it clear the under-tile in the course above at the head lap. The head lap varies from 6.5 to 7.5 cm, depending upon the pitch while the side lap is 5 cm. The taper in over-tile allows the tile in the next course to fit in. The ground work consists of rafters to which 5 cm × 2.5 cm battens are fixed at the *gauge* apart. Alternatively, 2.5 cm boarding, covered with felt may be used. The *gauge* equals the length of tile-lap. Vertical battens of size 2.2 cm × 7.5 cm are fixed between sides of adjacent under-tiles and to these the half-round over-tiles or boarding with 38 mm long copper nails, while the over-tiles are fixed to vertical battens with 75 mm nails.

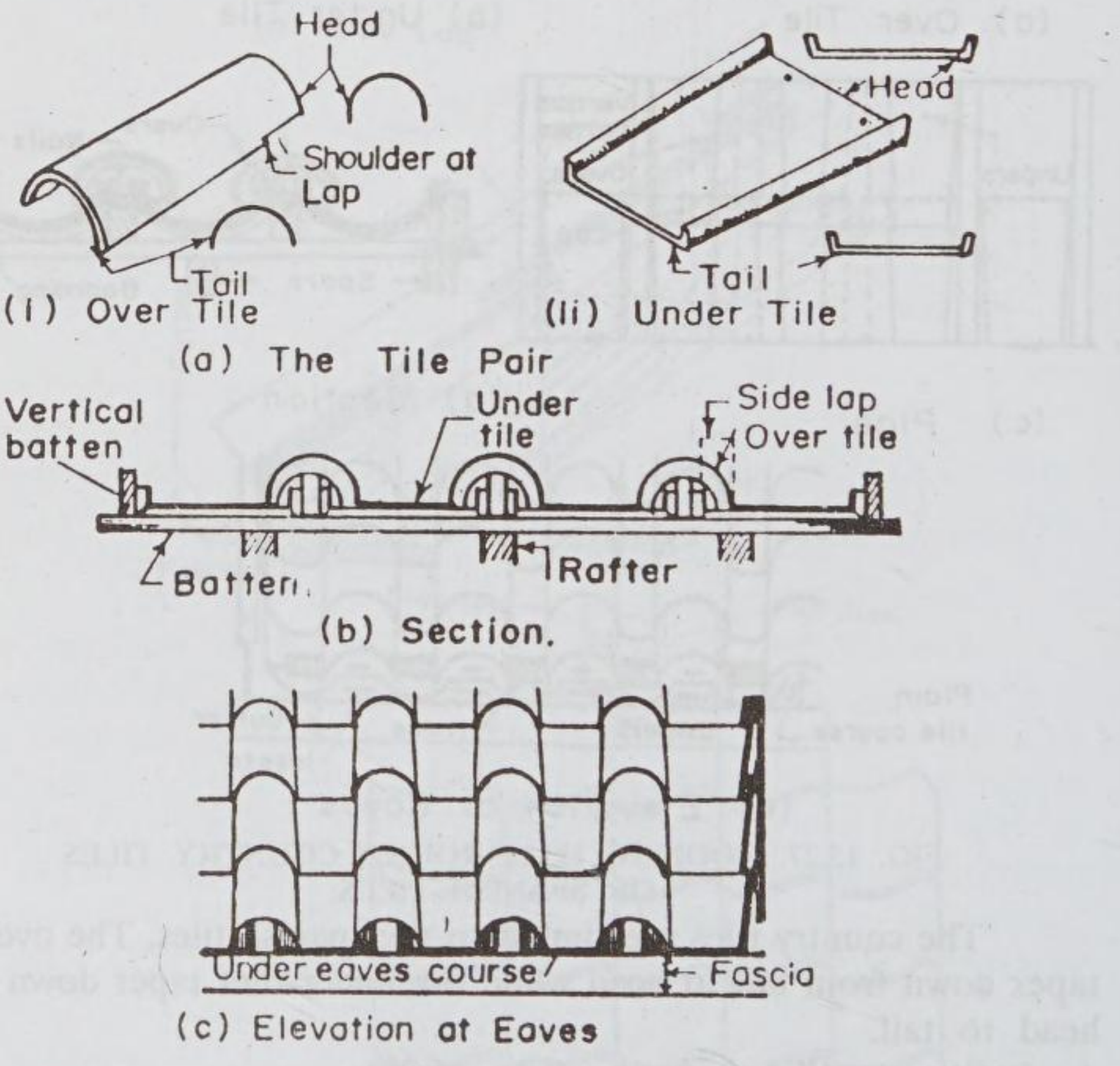


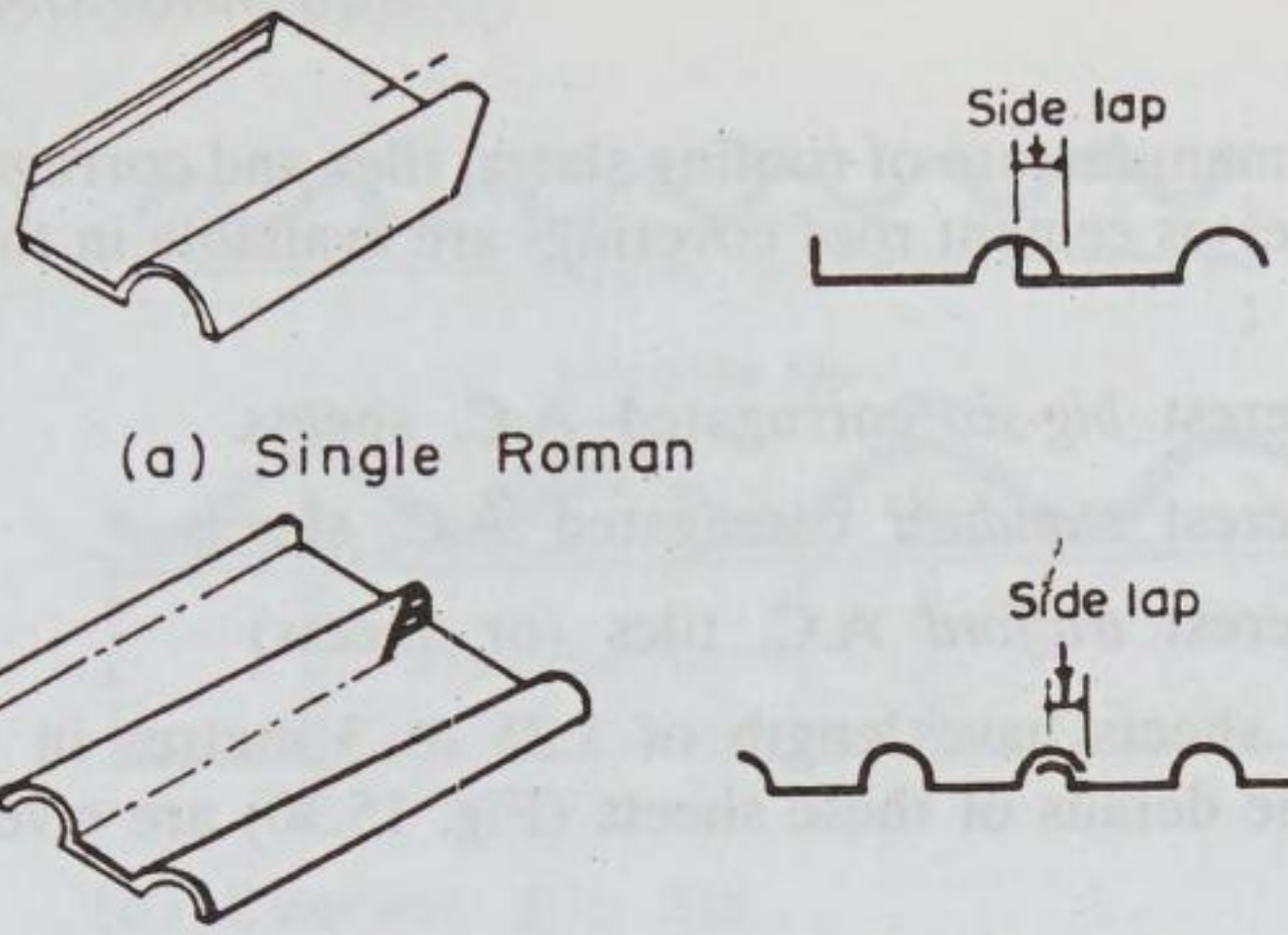
FIG. 15.28. ROOFING WITH ITALIAN TILES OR ALLAHABAD TILES.

5. Inter-locking tiles (Fig. 15.29)

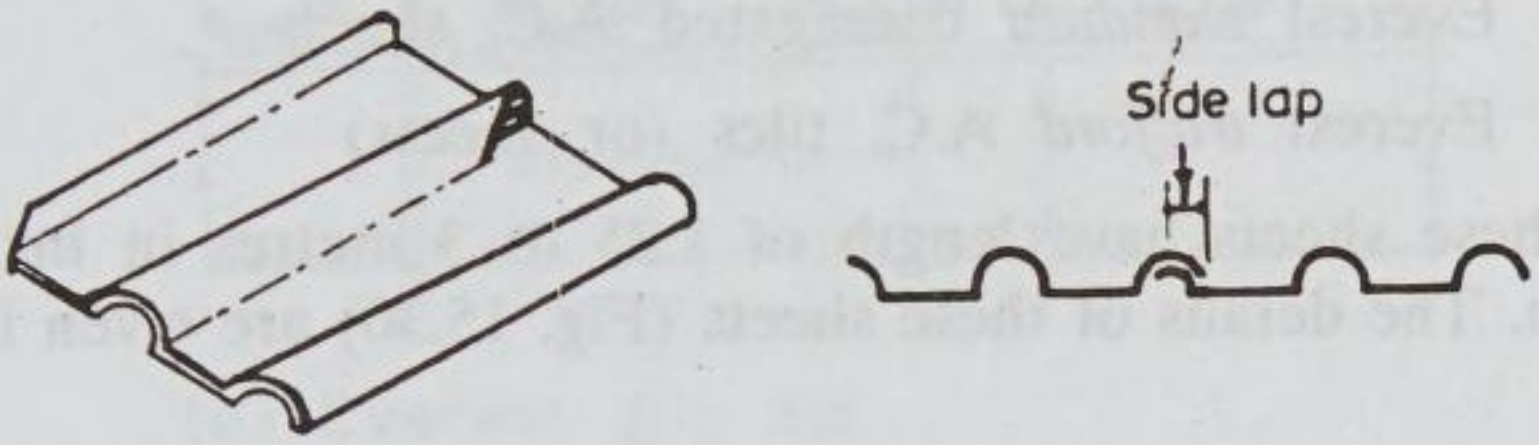
These tiles are available under patent names, with patent locking devices, the object of which is to prevent their dislodgment even in the most exposed conditions. These tiles are machine made. Some of the forms of inter-locking tiles are shown in Fig. 15.29.

***(d)* Asbestos cement sheets (A.C. sheets)**

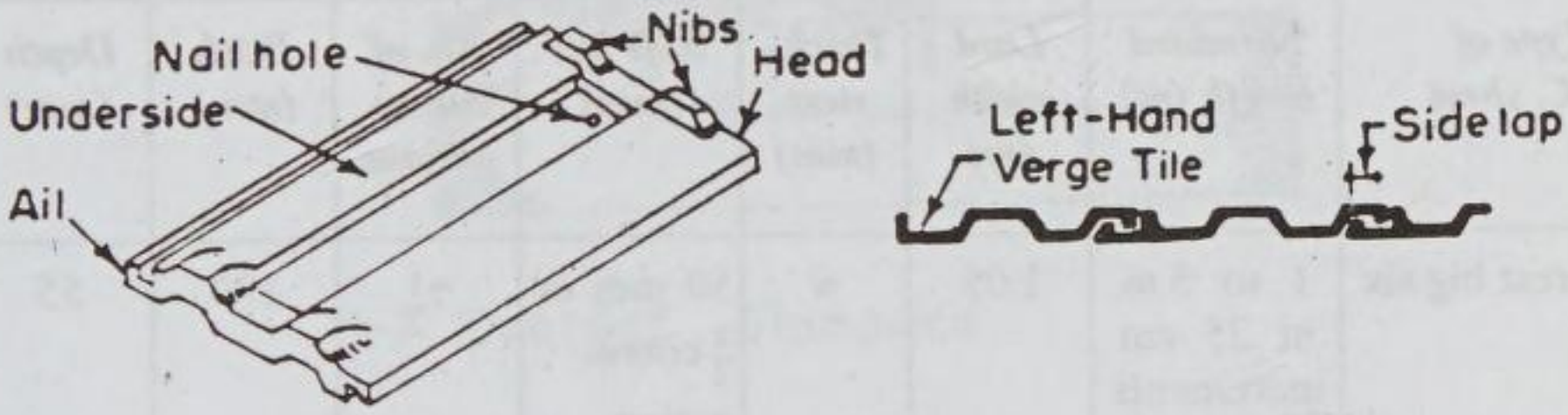
Asbestos cement sheets are now increasingly becoming popular



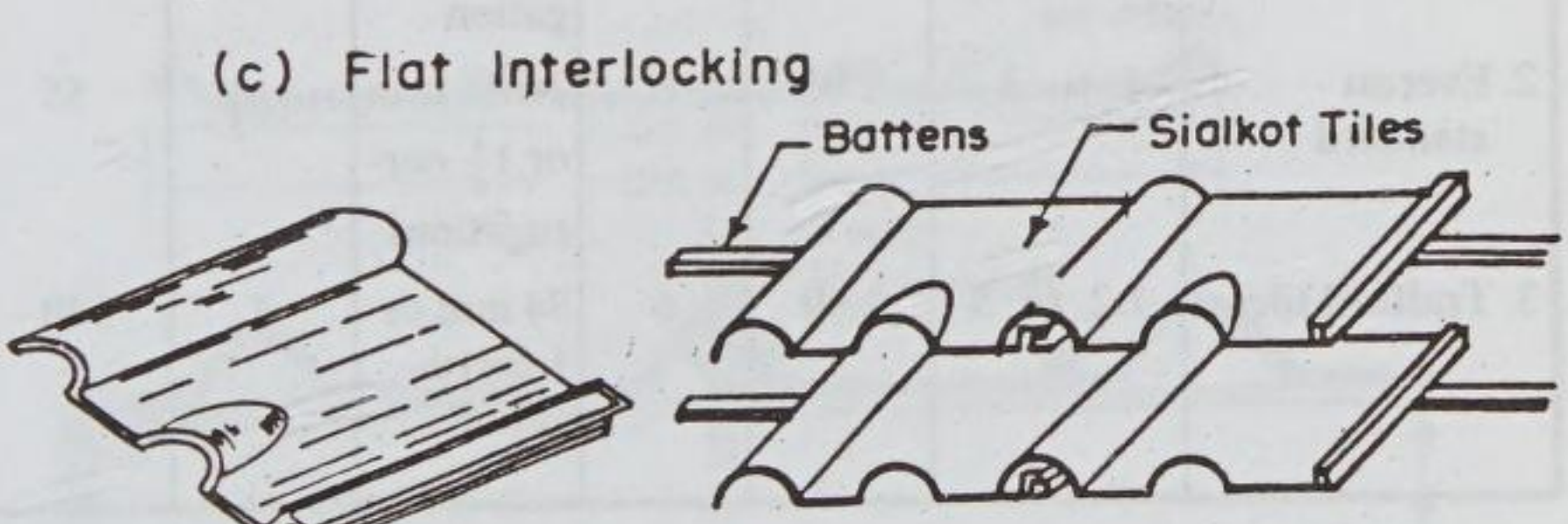
(a) Single Roman



(b) Doube Roman



(c) Flat Interlocking



(d) Sialkot Tiles

FIG. 15.29. VARIOUS FORMS OF INTER-LOCKING TILES.

for industrial buildings, factories, sheds, cinema houses, auditorium and even residential buildings, since they are cheap, light weight, tough, durable, water tight, fire-resisting and vermin resistant. The biggest advantage is that they are available in bigger units unlike tiles, and hence supporting frame work (ground work) is also cheaper, easier and lighter. These sheets do not require any protective paint, and no elaborate maintenance is required. Also, the construction with A.C. sheets is very fast. A.C. sheets are manufactured from asbestos, fibre (about 15%) and Portland cement. Asbestos is a silky fibrous mineral existing in veins of metamorphized volcanic rocks. It is found in several varieties but white asbestos, which is a compound of magnesia and silica, is principally used. Asbestos cement is now

used for the manufacture of roofing slates, tiles and corrugated sheets. In India, asbestos cement roof coverings are available in the following three forms :

1. Everest *big-six* corrugated A.C. sheets.
2. Everest *standard* corrugated A.C. sheets.
3. Everest *trafford* A.C. tiles (or sheets)

These sheets have length of 1.25 to 3 metres in increments of 15 cm. The details of these sheets (Fig. 15.30) are given in Table 15.1.

TABLE 15.1. PARTICULARS OF ASBESTOS CEMENT SHEETS

Type of A.C. sheet	*Standard length (m)*	*Laid width (m)*	*Thick-ness (mm)*	*Side lap (mm)*	*No. of corru-gations*	*Pitch (mm)*	*Depth (mm)*
1. Everest big six	1 to 3 m in 25 cm increments	1.05	6	50 mm or $\frac{1}{2}$ corru-gation	$7\frac{1}{2}$	130	55
2. Everest standard	1 to 3	1.05	6	100 mm or $1\frac{1}{2}$ cor-rugations	$10\frac{1}{2}$	55	25
3. Trafford tiles	1.2 to 3 m	1.09	6	74 mm or 1 corru-gation	4	340	50

The *big-six* type A.C. sheets have $7\frac{1}{2}$ corrugations per sheet and their overall depth is 55 mm. These sheets are fixed direct with smooth surface uppermost, to either steel purlins or wood purlins. The *standard corrugated sheet* is a smaller version of big-six, with over all depth of corrugation of 25 mm. There are $10\frac{1}{2}$ corrugations per sheet. The end or head lap is 150 mm and the side lap is equal to approximately $1\frac{1}{2}$ corrugations or 100 mm. *Trafford tiles* are large tiles of 1.09 m standard width. Each sheet has four 50 mm deep corrugations alternating with flat portions. They are fixed to steel purlins by 8 mm diameter hook bolts, or straight bolts, and to wood purlins by 115 mm long driving screws. The head lap is 150 mm and the side lap is approximately one corrugation of 74 mm.

Fig. 15.31 shows typical fixing bolts and screws used with corrugated A.C. sheets. Fig. 15.32 shows the details of fixing big-six A.C. sheet at eaves, ridge and intermediate locations.

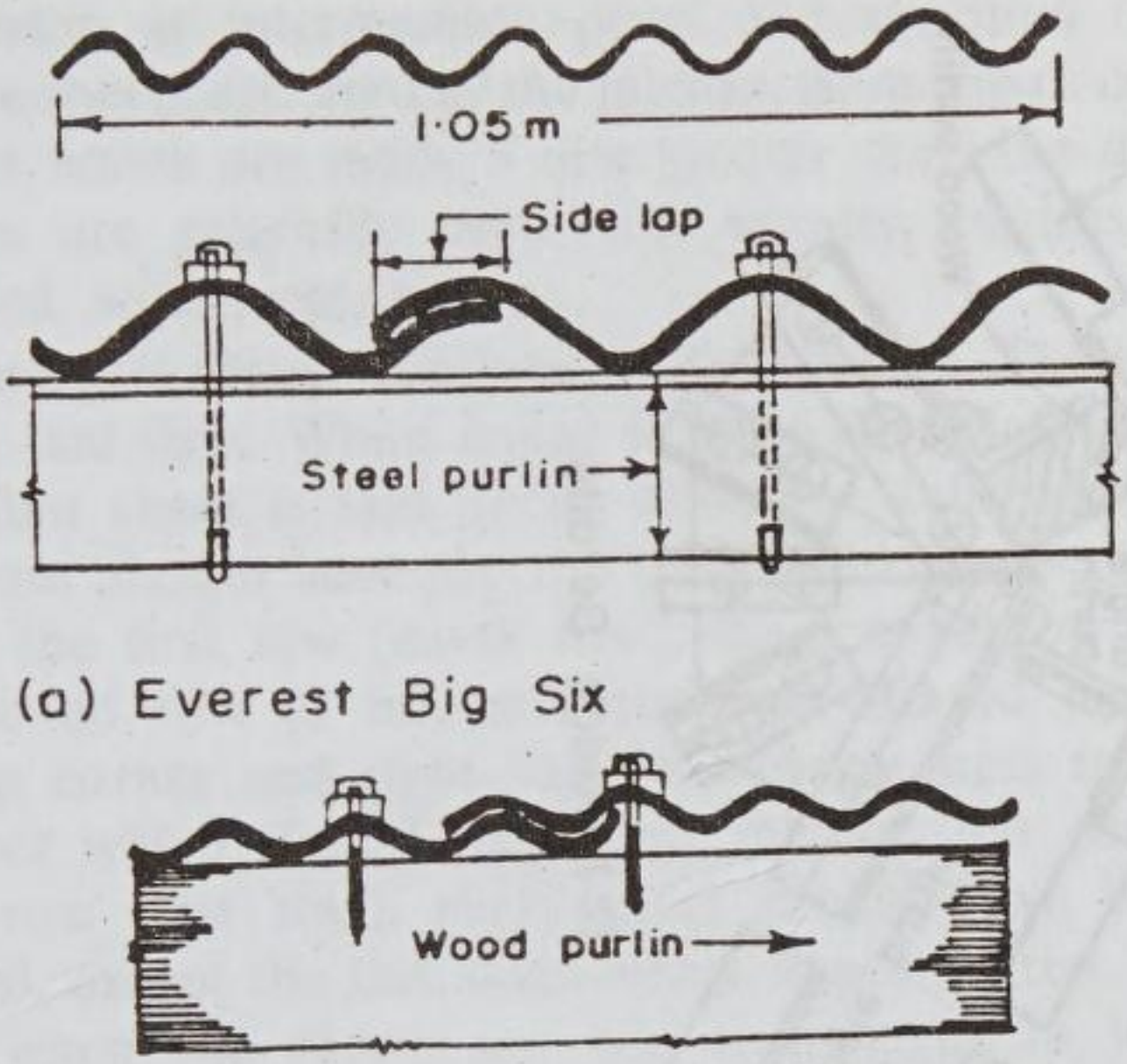


FIG. 15.30. A.C. SHEETS

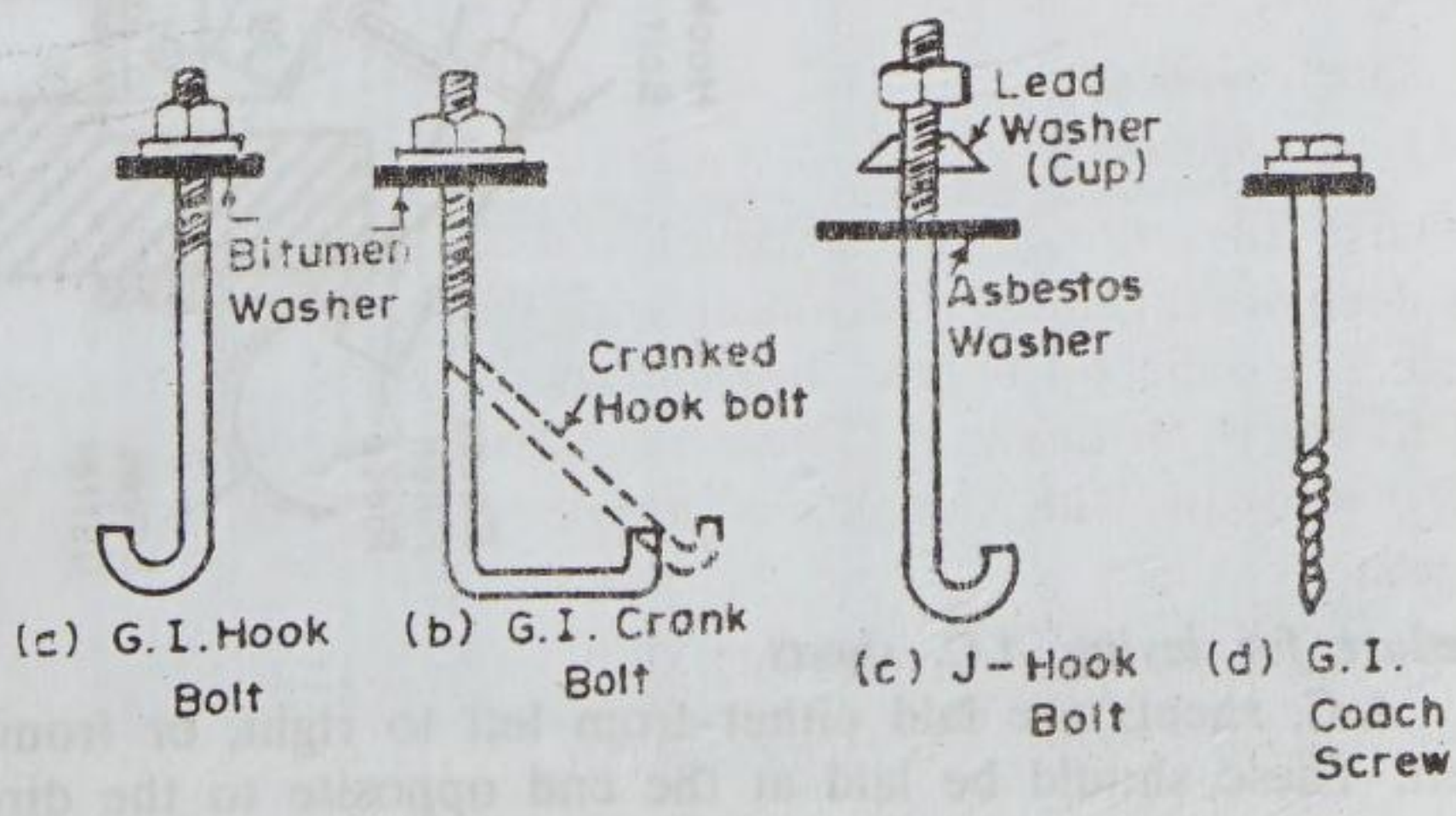


FIG. 15.31. FIXING BOLTS AND SCREWS.

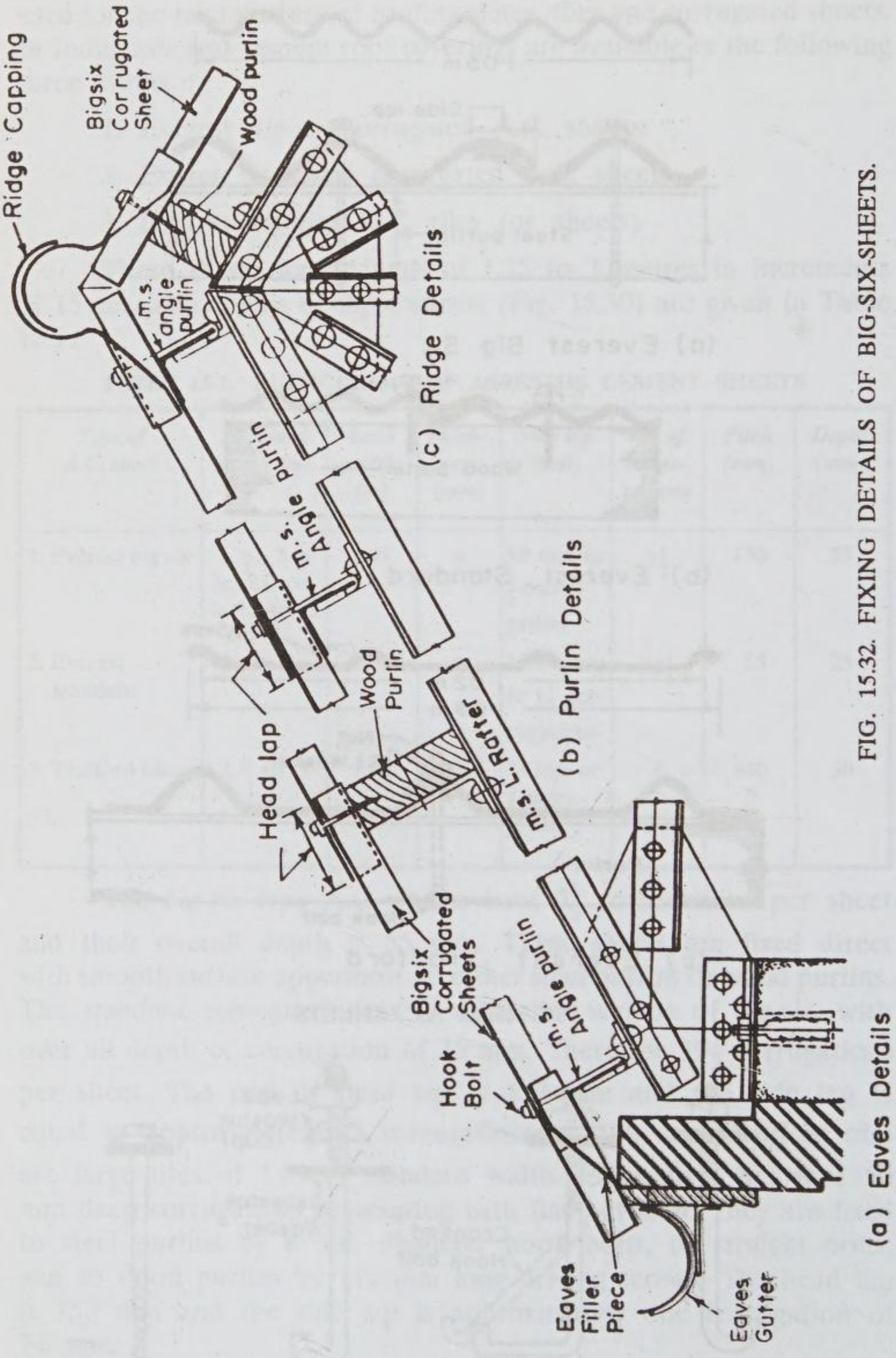


FIG. 15.32. FIXING DETAILS OF BIG-SIX-SHEETS.

Procedure for laying A.C. sheets

A.C. sheets are laid either from left to right, or from right to left. These should be laid at the end opposite to the direction

of prevailing wind and rain. The purlin spacing are adjusted to provide specified overlap at intermediate point and specified overhang at the eaves. The sheets are fixed to the purlins, from top of corrugations, through holes which are made 3 mm greater than the dia. of bolts. Coach screws are generally used with wooden purlins and crank bolts are used with steel purlins.

The laying is always commenced from eaves. The eaves course is, therefore, laid first. When laying is done from left to right (Fig. 15.33), the first sheet is laid uncut while the subsequent sheets in the bottom row should have the top left hand corners cut or *mitred.* After laying the first tow (eaves row), the second row is laid. The sheets in second row or intermediate rows should have both the left hand top corner and right hand bottom corners mitred, except the first sheet which should have only the top left corner mitred. In the top row (last row), every sheet should have bottom right corner mitred, except the last sheet which is not mitred. The process is reversed when the sheets are laid from right to left.

The following points should be noted while fixing A.C. sheets.

1. The A.C. sheets should be laid with smooth side upward, and the end marked 'Top' pointing toward the ridge.

2. End lap and side lap should be properly maintained. General end lap is 15 cm, but this can be varied to suit purlin spacing.

3. Purlin spacing and length of sheets should properly checked, before laying.

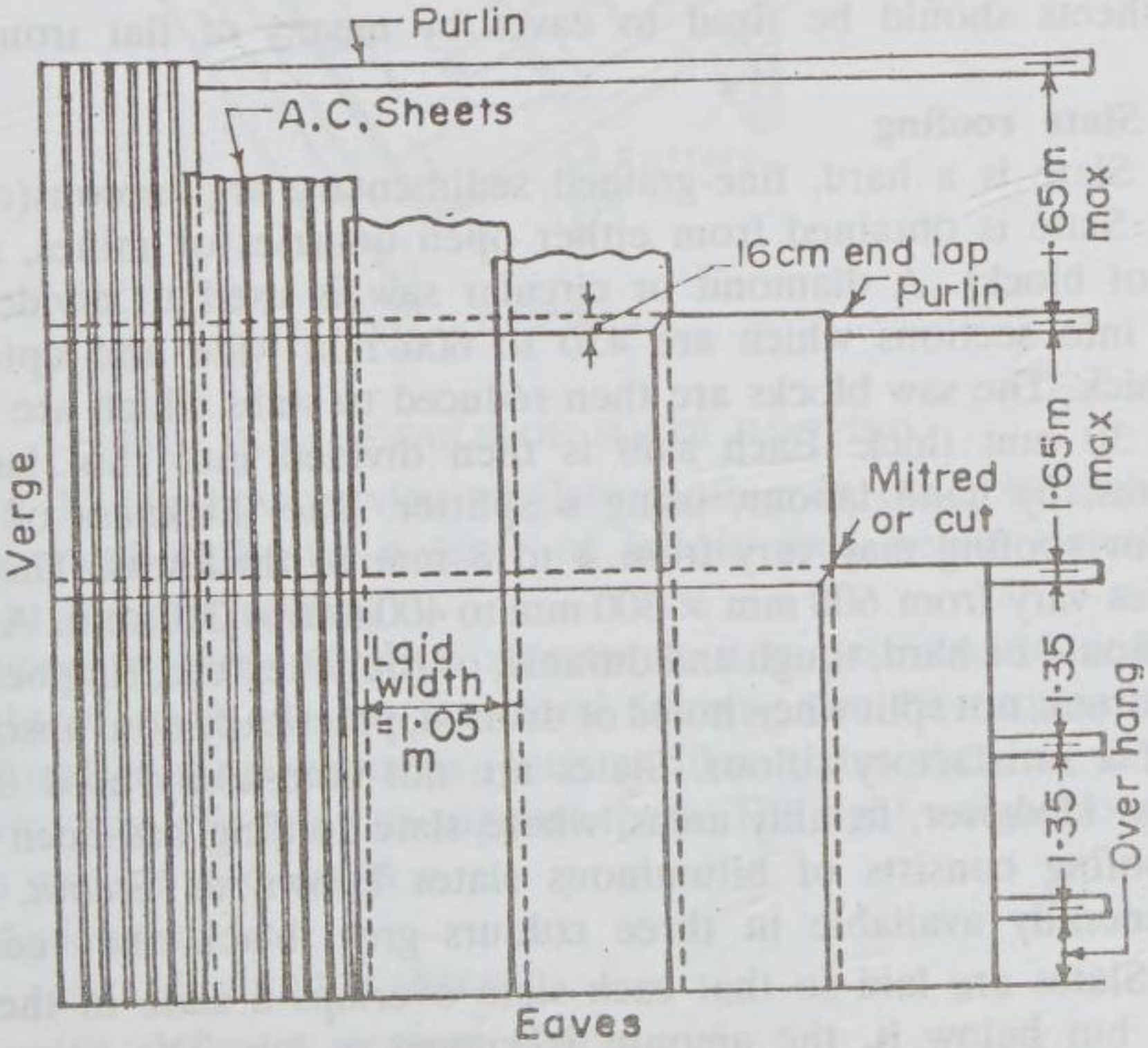


FIG. 15.33. LAYING OF A.C. SHEETS (LEFT TO RIGHT)

4. The holes for fixing accessories should be *drilled* (and not punched) in the crown of the corrugations. The diameter of the holes should be 3 mm greater than the diameter of the fixing bolt or screw. Thus 8 mm dia. drilled holes, and screwed lightly.

5. Bitumen washers should be provided under G.I. flat washer. The nuts of the screws or bolts are moderately tightened when 10 to 12 sheets have been laid. They should not be screwed very tight.

6. Ridge cappings should be secured to the ridge purlin.

7. The sheets should be 'mitred' properly as required.

8. The unsupported overhang of A.C. sheets should not exceed 30 cm.

(e) Galvanised Iron corrugated sheets (G.I. sheets)

G.I. sheets are also widely used. They are stronger than A.C. sheets. However, because of their higher cost, they are now gradually replaced by A.C. sheets. They are not used for slopes flatter than 1 in 4. G.I. sheets are manufactured with corrugations running from one end to the other. The corrugations impart additional strength to the sheets. G.I. sheets are made of iron sheets which are galvanised with zinc to protect them from rusting action of water and wet weather. These sheets are fixed in a manner similar to the A.C. sheets. End lap should not be less than 15 cm and the side lap varies from $1\frac{1}{2}$ to 2 corrugations. The holes are either drilled or punched in the sheet crowns. The sheets are secured to purlins by means of G.I. hook bolts, screws and nails etc., with curved washers. The sheets should be fixed to eaves by means of flat iron wind ties.

(f) Slate roofing

Slate is a hard, fine-grained sedimentary argillaceous(clayey) stone. Slate is obtained from either open quarries or mines, in the form of blocks. A diamond or circular saw is used to divide each block into sections which are 450 to 600 mm wide and upto 360 mm thick. The saw blocks are then reduced to slabs which are about 15 to 30 mm thick. Each slab is then divided into thin laminae or slates, by hand labour, using a splitter. The thickness of slate, used for roofing may vary from 4 to 8 mm in thickness. The sizes of slates vary from 600 mm × 300 mm to 400 mm × 200 mm. A good slate should be hard, tough and durable, of rough texture, ring bell-like when struck, not split when holed or dressed, practically non- absorbant and of a satisfactory colour. Slates are not commonly used in our country. However, in hilly areas, where slate roofing has been used, the roofing consists of bituminous slates known as *Eternit.* They are generally available in three colours–grey, black and red.

Slates are laid so that each slate overlaps a slate in the next course but below it, the amount is known as lap. The amount of

lap depends upon the pitch and the exposure. For fixing slates, two holes are made at the centre or the head. The holes are made from the bed of the slab so that the spalling forms a countersinking for the head of nail. Slates are fixed to the battens by means of copper or zinc nails. The spacing of the battens, known as gauge is determined from the following expression.

$$\text{Gauge} = \frac{\text{length of slate} - \text{lap}}{2}$$

Ridges and hips are generally covered with blue or grey ridge tiles—matching the colour of slate.

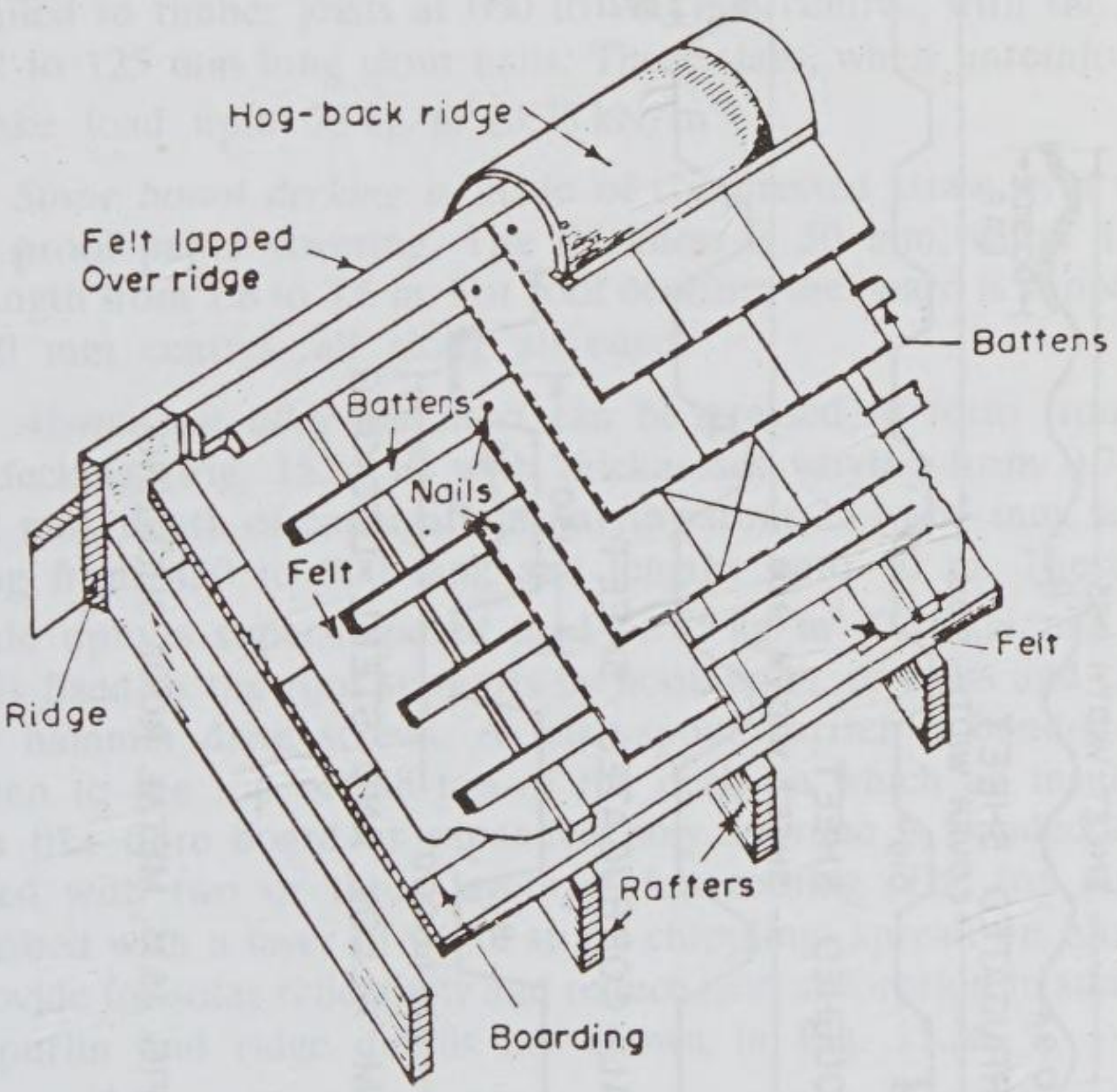


FIG. 15.34. DETAILS OF SLATE ROOFING.

Fig. 15.34. shows a view of slate roofing. In order to exclude rain water and moisture, a layer of felt is used below slates.

(g) Light Weight roofing

For wide-span industrial structures, it is desirable to reduce the weight of roof, so that structural framing can be economised. Conventional roofing materials (such as tiles, slates etc.) are heavy and require heavy framing to support them. The light weight roofing materials are of two types :

(*a*) ***Sheeting***

(*i*) Aluminium sheets

(*ii*) Asbestos cement sheets.

(*b*) ***Decking***

(*i*) Wood wool

(*ii*) Straw board

(*iii*) Aluminium alloy and steel decking.

All these require a water proof layer of asphalt or roofing felt. Sheeting is used for sloping roofs while decking is used both for sloping as well as flat roofs.

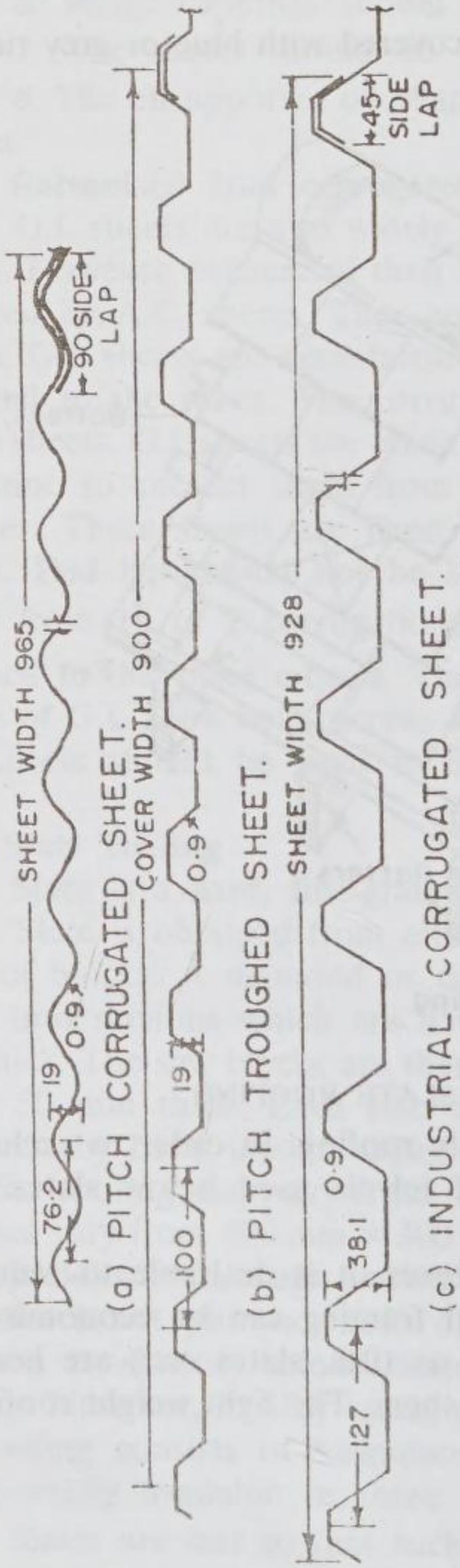


FIG. 15.35. METAL ROOF SHEETING.

Aluminium roof sheeting consists of aluminium alloyed with a small percentage of manganese for strength. It is the lightest of all roofing. The sections are shown in Fig. 15.35.

Wood wool is made from wood fibre interwoven together and cement bonded under pressure in a mould. They are available in the form of slabs, varying in thickness from 12 mm to 100 mm, and in size of 0.6 m width and upto 3.9 m length. Wood wool has good sound absorbing and thermal insulation properties. For roofing, the slabs are generally of 50 to 75 mm thickness. They are nailed to timber joists at 600 to 900 mm centres, with the help of 102 to 125 mm long clout nails. These slabs, when unreinforced, can take load upto 75 kg/m^2 (0.75 kN/$m^{2)}$

Straw board decking is made of compressed straw with thick water proof paper covering. The thickness is 50 mm, width 1.2 m and length from 1.8 to 3.6 m. For roof decking, the board is supported at 600 mm centres, all along all edges.

Aluminium alloy and steel can be pressed to form troughed roof decking (Fig. 15.35 *d*) with thicknesses varying from 0.7 mm to 1.2 mm, depth of corrugations varying from 25 to 85 mm, widths varying from 450 to 900 mm, and lengths upto 10 m. These are suitable upto a super-imposed load of 75 kg/m^2 (0.75 kN/m^2). The deck is fixed to the roof supports by hook bolts, or bolts and cleats, or by hammer drive screws. A felt vapour barrier is bonded with bitumen to the top of the top of the deck on which an insulating media like fibre board or expanded poly stryrene is bonded to be covered with two or three layers of felt roofing. The top surface is finished with a layer of white stone chipplings spread on bitumen to provide for solar reflectivity and reduce heat absorption in summer. The purlin and ridge details are shown in Fig. 15.36.

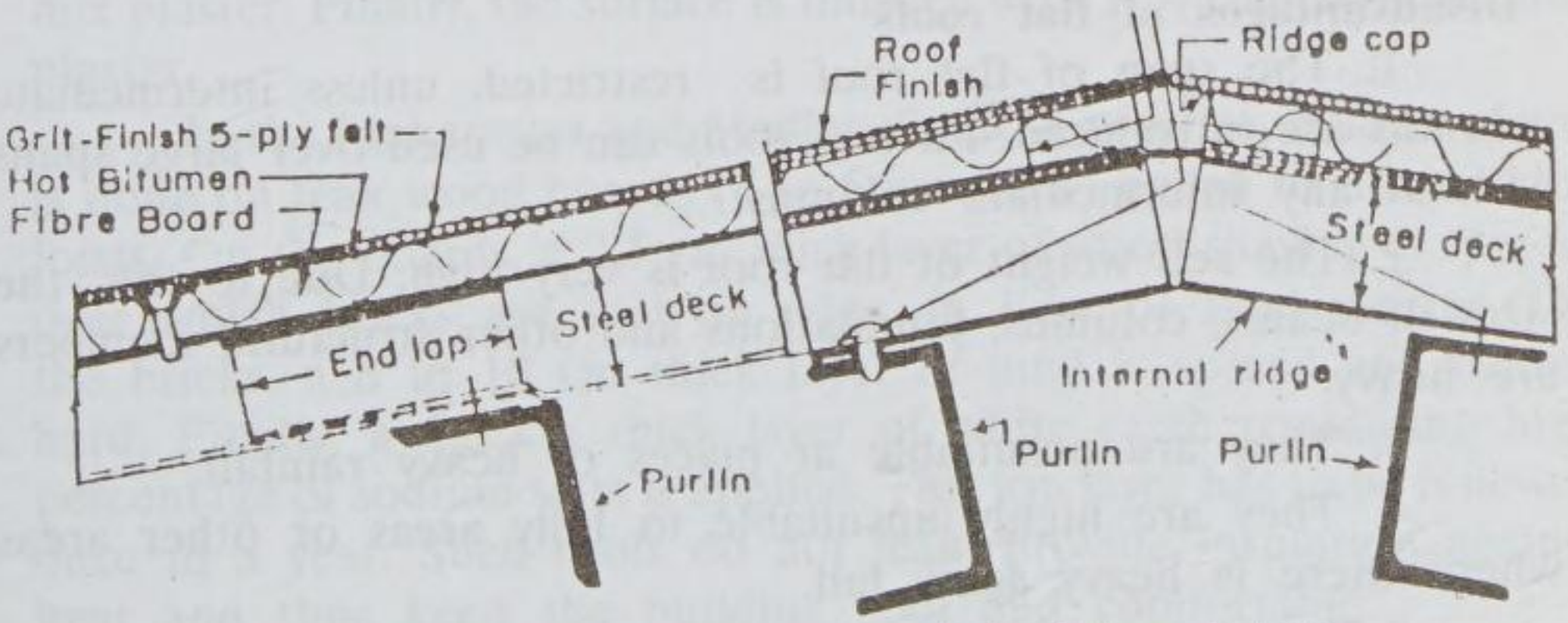


FIG. 15.36. ALUMINIUM ALLOY AND STEEL DECKING

15.10. FLAT TERRACED ROOFING

Flat roof is the one which is either horizontal, or practically horizontal with slope less than 10°. Even a perfectly horizontal roof has to have some slope at top so that rain water can be drained off easily and rapidly. Similar to the upper floor, the flat roofs can be constructed of flag stones, R.S.J. and flag stones, reinforced cement concrete, reinforced brick work, jack arch roof or precast cement concrete units. However, the flat roof differ from the upper floor only from the point of view of top finish, commonly called terracing, to protect it from adverse effects of rain, snow, heat etc.

Advantages of flat roofs

1. The roof can be used as terrace for playing, gardening sleeping and for celebrating functions.
2. construction and maintenance is easier.
3. They can be easily made fire proof, in comparison to pitched roof.
4. They avoid the enclosure of the triangular space. Due to this, the architectural appearance of the building is very much improved.
5. Flat roofs have better insulating properties.
6. They require lesser area of roofing material than pitched roof.
7. They are more stable against high winds.
8. They do not require false ceiling, which is essential in pitched roofs.
9. Flat roofs are proved to be overall economical.
10. In multi-storeyed buildings, flat roof is the only choice, since overhead water tanks and other services are located on the terrace.
11. The construction of upper floors can be easily done over flat roofs, if so required in future.

Disadvantages of flat roofs

1. The span of flat roof is restricted, unless intermediate columns are introduced. Pitched roofs can be used over large spans without any intermediate columns.
2. The self weight of flat roof is very high. Due to this, the sizes of beams, columns, foundations and other structural members are heavy.
3. They are unsuitable at places of heavy rainfall.
4. They are highly unsuitable to hilly areas or other areas where there is heavy snow fall.
5. They are vulnerable to heavy temperature variations, specially in tropics, due to which cracks are developed on the surface. These cracks may lead to water penetration later, if not repaired in time

6. It is difficult to locate and rectify leak in flat roof.

7. The speed of flat roof construction is much slower than the pitched roof.

8. The initial cost of flat roof is more than pitched roof.

9. The flat roof exposes the entire building to the weather agencies, while the projecting elements (such as eaves etc.) of pitched roof provide some protection to the building.

Types of flat terraced roofing

Following are the commonly used terraced roofing :

1. Mud-terrace roofing.
2. Brick-jelly or Madras terrace roofing.
3. Mud-phaska terracing with tile paving.
4. Lime concrete terracing.
5. Lime concrete terracing with tile paving.
6. Bengal terrace roofing.
7. Light weight flat roofing.

1. Mud-terrace roofing

This type of terracing is suitable where rainfall is less. It can be provided either on tiles (Punjab type terracing) or on wood boards (Maharastra and Madhya Pradesh practice). In both the cases, terracing is made with white earth mud containing large percentage of sodium salt.

The mud-terracing in Punjab is provided over roof which consists of 50 mm × 50 mm× 6 mm T-sections spaced at 32 cm centre to centre over R.S.J. Well-burnt tiles of size 30 cm × 30 cm × 5 cm or 30 cm × 15 cm × 5 cm are placed between the flanges of the T-sections ; using lime mortar. Over the tiles, a 15 cm thick layer of stiff mud, white in colour and containing sodium salts, is spread and beaten with sticks till the surface becomes hard and the beater rebounds. The surface is then plastered with mud and cow-dung mix plaster. Finally, the surface is finished with 1 : 4 cement-cow-dung plaster.

In the Maharastra and Madhya Pradesh practice, mud terracing is done on teak wood boards (4 to 5 cm thick) nailed to the wooden joists. On the boards, a 2.5 cm thick layer of wood shaving is spread, over which bricks are laid on edge, in lime or mud mortar. On the bricks, a 8 to 10 cm thick layer of mud is spread and beaten hard. Finally, a 2.5 cm thick layer of white earth containing high percentage of sodium salts is applied. This top layer has to be renewed once in a year. Such roofs do not leak, provide insulation against heat and thus keep the building cool and comfortable.

2. Brick jelly roofing or Madras terrace roofing

Fig. 15.37 shows the section through the roofing, which is

constructed in the following steps :

(*i*) Wooden joists are placed on R.S.J. with a furring piece in-between. The furring piece height at the centre is so adjusted that the required slope of the roof is obtained.

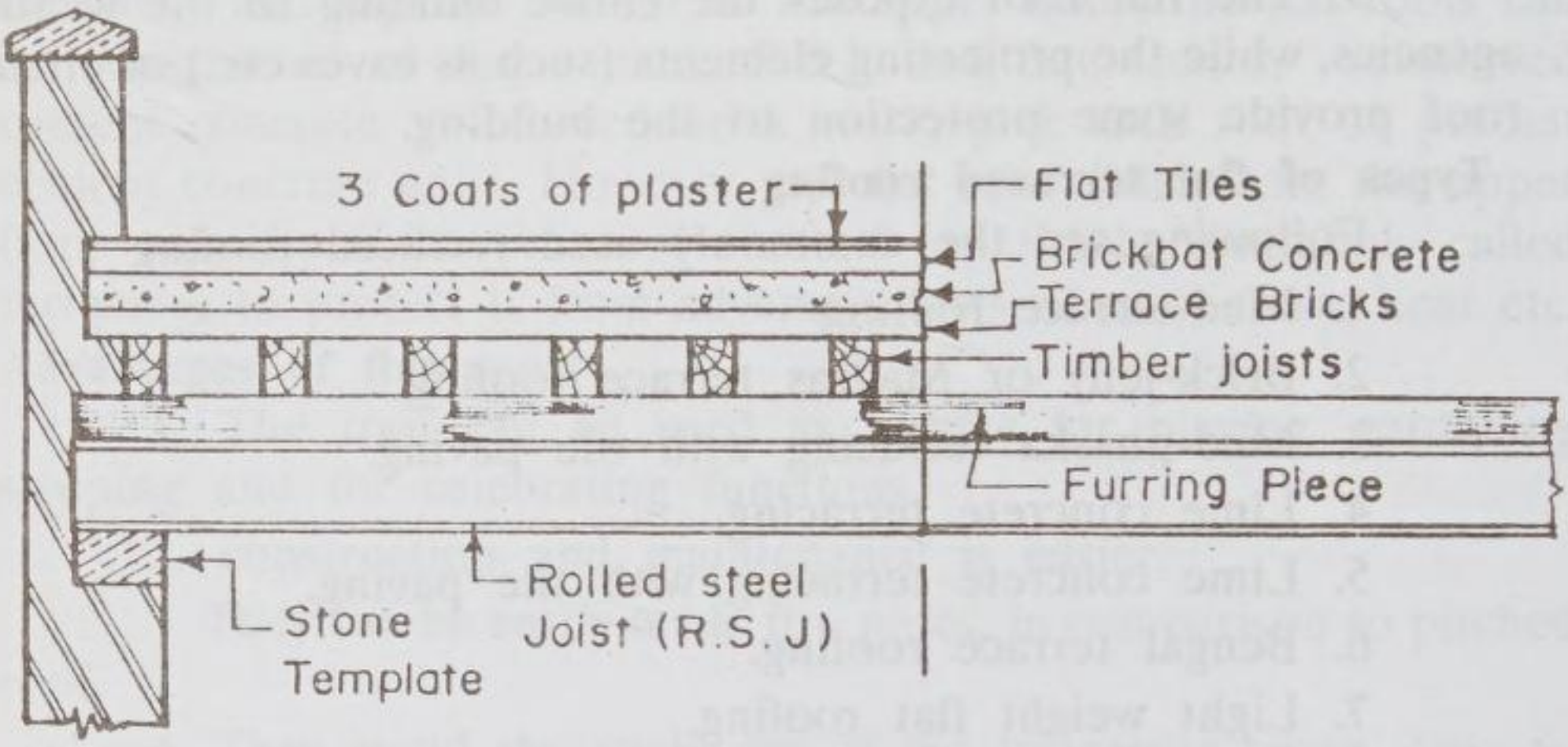


FIG. 15.37. MADRAS TERRACE ROOF.

(*ii*) A course of specially prepared bricks of size 15 cm× 5 cm×12 mm is placed on edge in lime mortar (1 : 1.5) laid diagonally across the joists.

(*iii*) After the brick course is set, a 10 cm thick layer of brick-bat concrete is laid, consisting of 3 parts of brick-bats, 1 part of gravel and sand, and 50 percent of of lime mortar by volume. The concrete is well-rammed for 3 days, so that the thickness reduces to 7.5 cm, by wooden hand beaters. The surface is cured for 3 days, by sprinking lime water.

(*iv*) When the brick-bat concrete has set, three courses of Madras flat tiles(15 cm × 10 cm × 12 mm) are laid in lime mortar (1 : $1\frac{1}{2}$), making a total thickness of 50 mm. The vertical joints of the tiles in successive layers should be broken. The joints of tiles in top layer are left open to provide key for top plaster. Alternatively, China mosaic tiles may be used.

(*v*) Finally, the top surface is plastered with three coats of lime mortar. The surface is rubbed and polished.

3. Mud-phuska terracing with tile paving

This method of terracing is equally suitable to hot as well as arid regions, and is commonly used over R.C.C. roofing. The section of roofing is shown in Fig. 15.38. The work is carried out in the following steps :

1. The R.C.C. slab is cleaned off dust and loose material. A layer of hot bitumen is spread over it at the rate of 1.70 kg of bitumen per square metre of roof surface.

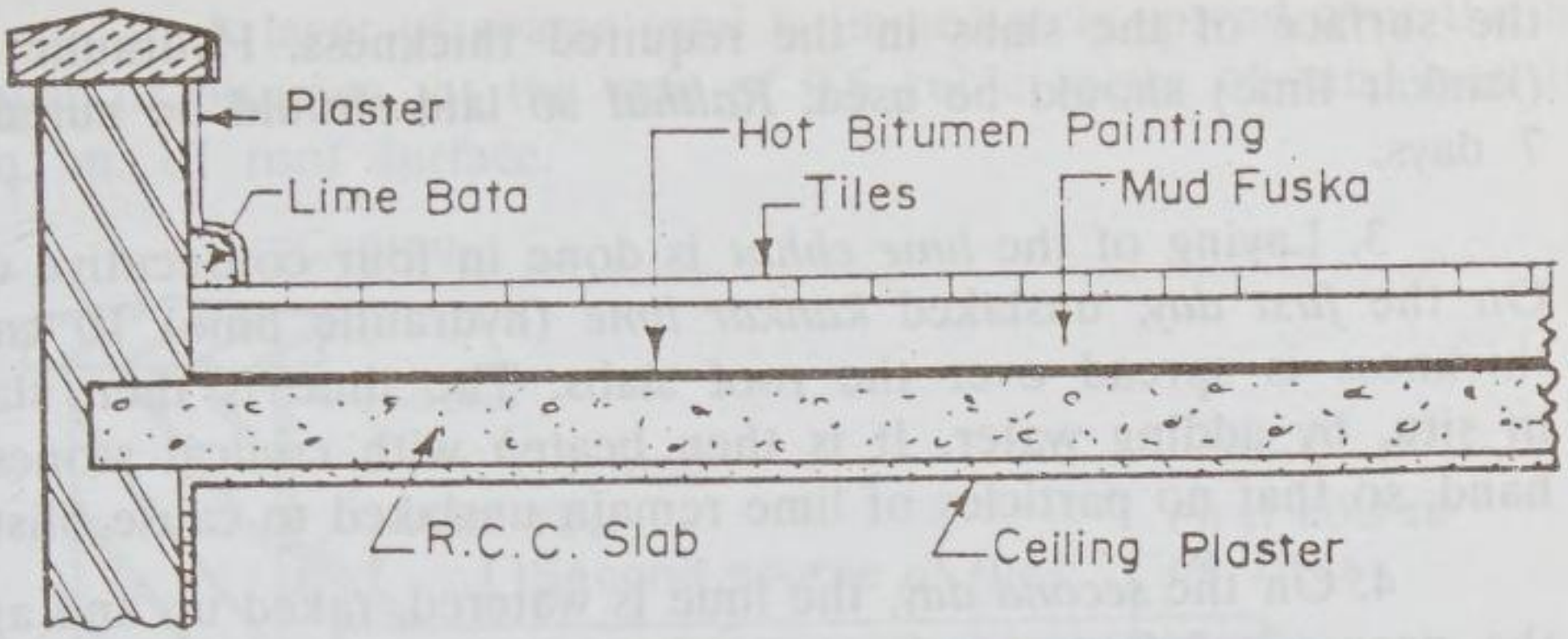


FIG. 15.38. MUD-PHUSKA AND TILE TERRACING

2. A layer of coarse sand is immediately spread over the hot coat of bitumen, at the rate of 0.6 m^3 of sand per 100 m^3 of roof surface.

3. Mud-phuska is prepared from puddled clay mixed with bhusa at the rate of about 8 kg of bhusa per m^3 of clay. A 10 cm thick layer of this mud-phuska is applied over the sand-bitumen layer. Proper slope (usually 1 in 40) is given in mud-phuska layer. Alternatively, slope may be given in R.C.C. slab itself.

4. The mud-phuska layer is consolidated properly. It is then plastered with 13 mm coat of mud-cow-dung mortar (3 : 1).

5. Tile bricks are laid flat on plastered surface. The joints are grouted in 1 : 3 cement mortar.

4. Lime concrete terracing : Jodhpur type roofing

This type of terracing is commonly used over flag stone roofing, though it can also be used over R.C.C. slab. The procedure of lime terracing varies from place to place. The one adopted for Jodhpur stone slab roofing is described below, in steps :

1. The longitudinal joints between the stone slabs are first pointed in cement mortar. The joints should be V-shaped, not exceeding 25 mm at the top and 10 mm at the bottom. This joint is filled with cement mortar(mix 1 : 2 to 1 : 4) and picked with stone chips of wedge shape and top finish rounded with cement mortar so as to project little above the slabs. Before filling mortar in the joints, flat strips of timber (or 3 inch dia. bamboos) should be kept along the joint on the other face of the stone slabs so that mortar does not fall down. Similarly, the space left over the walls at the ends of the slabs, and also the space on walls between the slabs where roof is continuous should be filled with 1 : 2 : 4 cement concrete. These joints should properly cured, at least for 7 days.

2. In order to provide proper slope to the roof, *ralthal* is laid. This is done by laying stone spawls in 1 : 2 lime mortar over

the surface of the slabs in the required thickness. Hydraulic lime (kankar lime) should be used. *Ralthal* so laid should be cured for 7 days.

3. Laying of the *lime chhat* is done in four consecutive days. On the *first day,* unslaked *kankar lime* (hydraulic lime) 10 cm in thickness is spread over the roof slabs. The lime is then slaked in situ, by adding water. It is then beaten with conical stones by hand, so that no particles of lime remain unslaked to cause blisters.

4. On the *second day,* the lime is watered, raked up and again the process of first day is repeated.

5. On the *third day,* 250 gm of hemp (finely choped) and methi 750 gm finely powdered per 10 sq. metre is evenly and thoroughly mixed with the lime. Then coarse stone aggregate duly washed should be spread over this lime in a thickness not less than 10 cm.

The coarse aggregate is thoroughly beaten with conical stones by hand so that this stone aggregate gets well-embedded in lime mass.

6. On the fourth day, stone grit or screening is spread in a layer of 40 mm and beaten with stone beaters till they are well set. This process of beating should continue with wooden thapies and by sprinkling water till the whole mass becomes stiff and offers resistance to penetration. Thickness of lime chhat should not be less than 15 cm at any place.

7. The above work should be cured at least for 7 days.

8. After seven days, *sandala coat* consisting of cream of lime is laid over the lime chhat in thin layers and rubbed for full four hours or more, using rounded pebbles for rubbing and polishing. During the process of rubbing, solution of 65 gm of Gur and 250 gm of Gugal per 10 square metres is sprinkled every now and then.

9. The surface thus prepared is cured with water at least for 15 days using damp sand or moist gunny bags so as to keep the surface constantly wet.

5. Lime concrete terracing with tiles

This type of terracing is commonly adopted over R.C.C. roofing. Fig. 15.39 shows a typical section of roofing, which is laid in the following steps :

1. The R.C. slab is cleaned off dust etc., land layer of hot bitumen is applied at the rate of 1.7 kg per sq. m. of roof surface.

2. A layer of coarse sand is immediately spread over the hot layer of bitumen, at the rate of 0.6 cubic metre of sand per 100 sq. m. of roof surface.

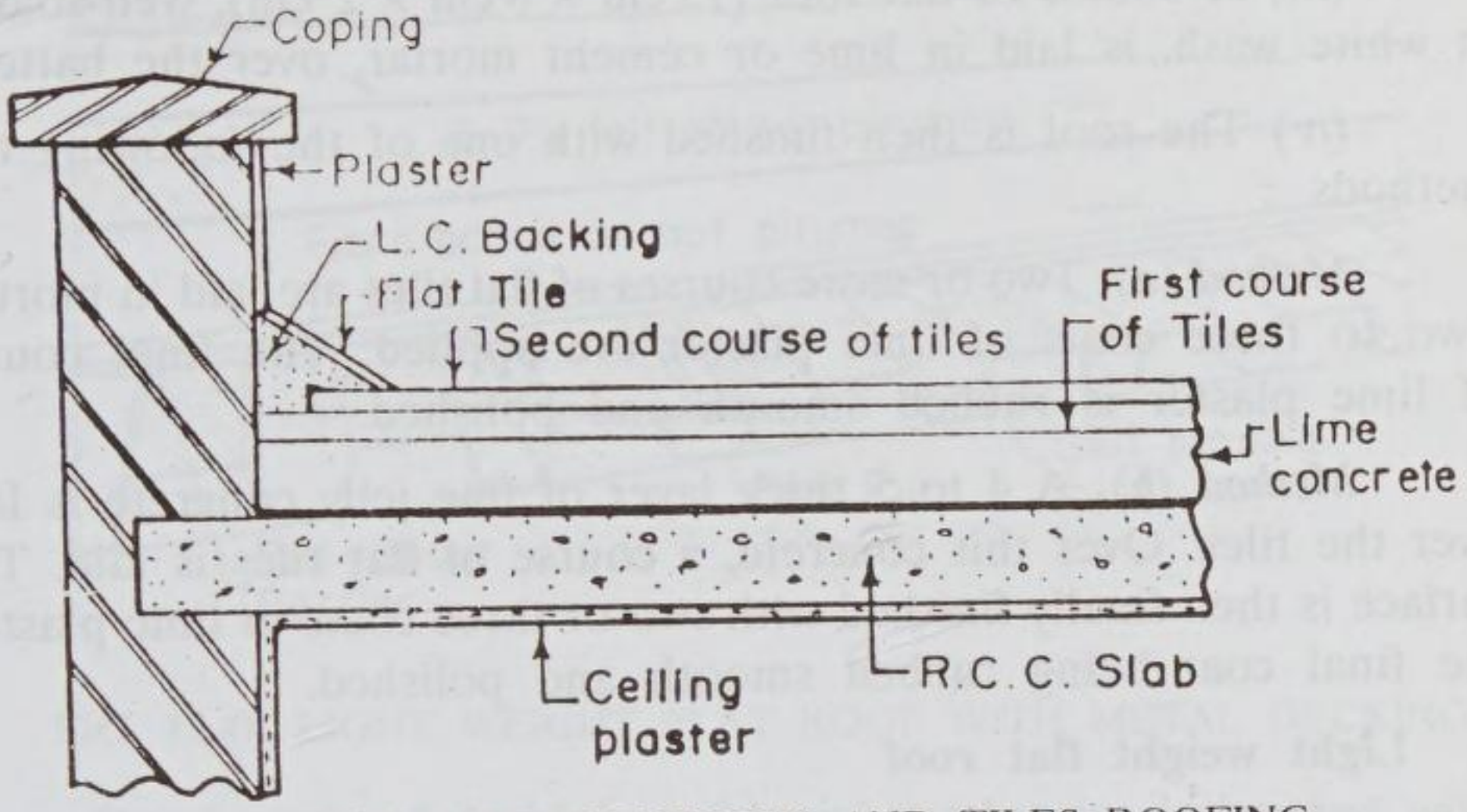


FIG. 15.39. LIME CONCRETE AND TILES ROOFING.

3. A 10 cm thick (average) layer of lime concrete is laid, in proper slope. The entire slope is given in lime concrete itself. The lime concrete may consist of 2 parts of lime, 2 parts of surkhi and 7 parts of brick ballast of 25 mm gauge. The concrete is well beaten.

4. Two courses of flat brick tiles are laid in 1 : 3 cement mortar. The joints of top course are pointed with 1 : 3 cement mortar. The vertical joints in the two courses are broken.

6. Bengal terrace roofing

This type of roofing is adopted for timber roofs of verandah etc. Fig. 15.40 shows the section of such roofing, which is constructed in the following steps :

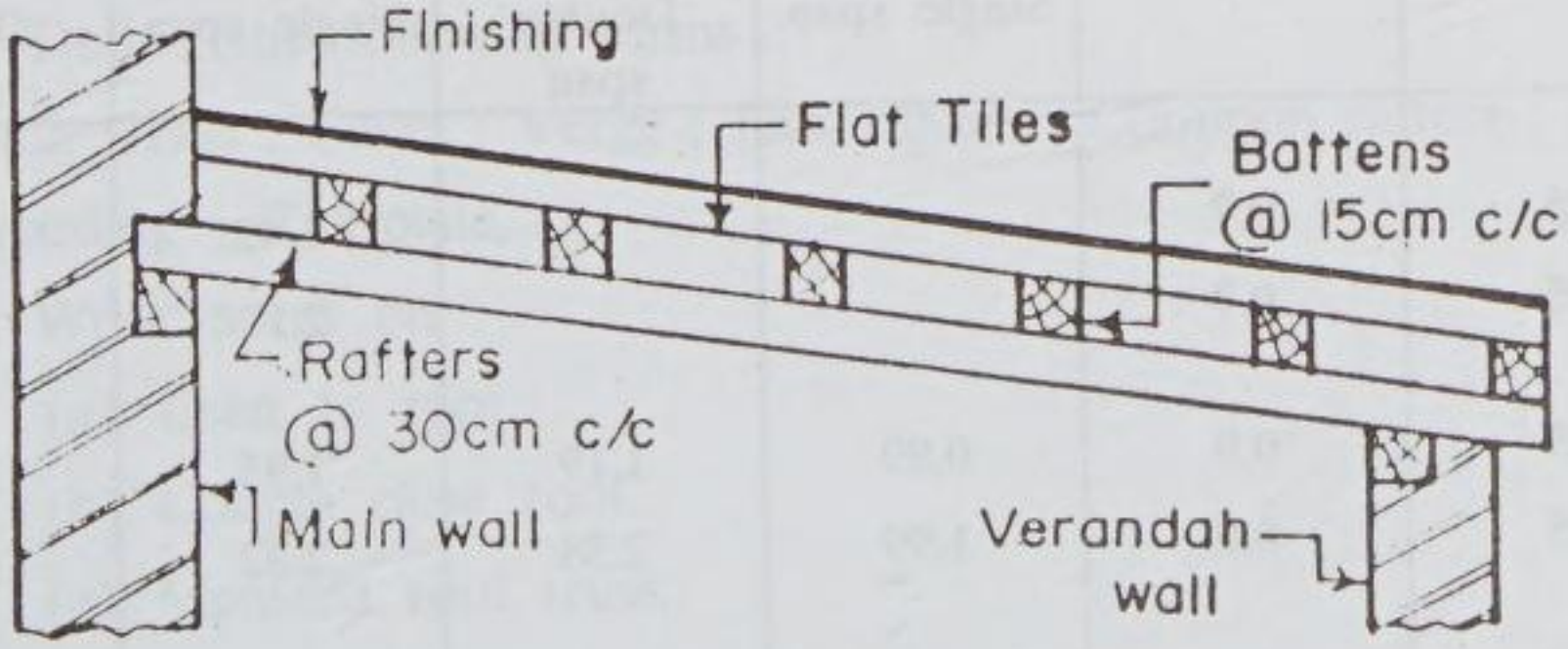


FIG. 15.40

(*i*) Wooden rafters are placed at 30 to 50 cm c/c, on some slope.

(*ii*) Wooden battens (5 × 1 cm) are placed across the rafters, at 15 c/c.

(*iii*) A course of flat tiles (15 cm × 8 cm × 2 cm), well-soaked in white wash, is laid in lime or cement mortar, over the battens.

(*iv*) The roof is then finished with one of the following two methods :

Method (*a*). Two or more courses of flat tiles are laid in mortar. Two to three coats of lime plaster are applied. The final course of lime plaster is rubbed smooth and polished.

Method (*b*). A 4 to 5 thick layer of fine jelly concrete is laid over the tiles. Over this concrete, a course of flat tiles is laid. The surface is then finally finished with two or three coats of lime plaster, the final coat being rubbed smooth and polished.

7. Light weight flat roof

This consists of aluminium alloy and steel decking, described earlier under pitched roofing. The section of roof with aluminium alloy and steel decking is shown in Fig. 15.41. The decking shown has an additional soffit sheet. The decking sheet is suitably supported on steel beams. Table 15.2 gives the maximum span over which these can be used, for an imposed load of 75 kg/m^2 (0.75 kN/m^2).

TABLE 15.2. METAL DECKING

Depth of corrugation (mm)	*Thickness of metal (mm)*	*Maximum span (m)*			
		Aluminium		*Steel*	
		Single span	Double span	Single span	Double span
25	0.7			1.42	1.71
45	0.7			2.1	2.54
25	0.9	0.99	1.19	1.95	2.31
45	0.9	1.99	2.38	2.82	3.34
25	1.2	1.54	1.85		
45	1.2	2.25	2.67		
85	1.2	3.60	3.98		

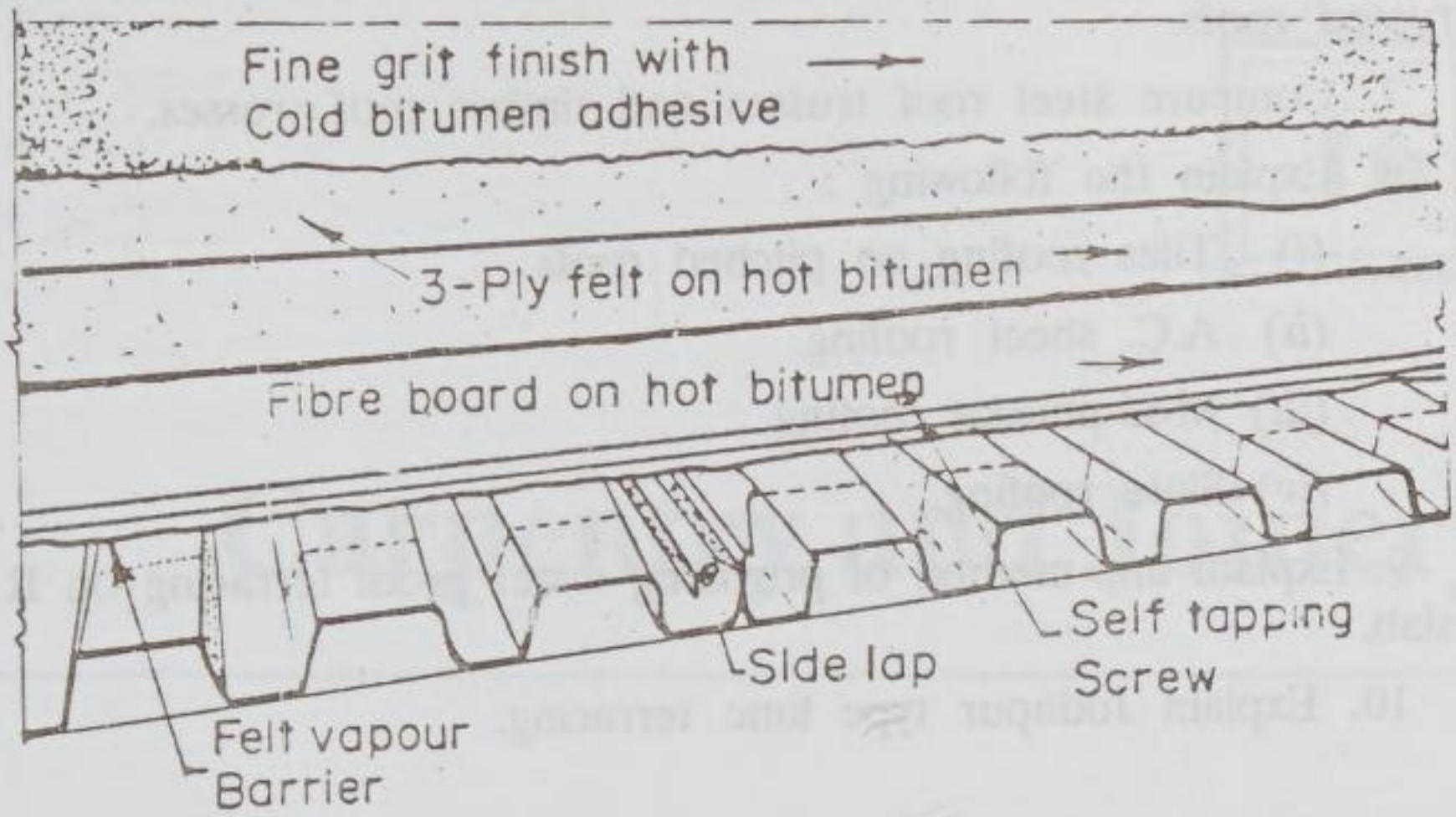


FIG. 15.41. LIGHT WEIGHT FLAT ROOF WITH METAL DECKING

On the top of decking, a felt vapour barrier is bonded with bitumen. Over it, fibre board or expanded polystyrene is bonded, for insulation. This is then covered with two or three layers of felt roofing. Finally, the top surface is finished with a layer of white stone chipping spread on bitumen to provide for solar reflectivity and reduce heat absorption.

PROBLEMS

1. (*a*) State briefly the essential requirements of a good roof.
   (*b*) Compare merits and demerits of flat and pitched roofs.
2. Explain, in brief, but with sketches, various basic forms of pitched roofs.
3. Define the following terms :

Pitch ; Hip ; Eaves ; Verge ; Jack rafters ; Common rafters ; Cleat; Boarding ; Template.

4. Write notes on :
   (*a*) Lean to roof.
   (*b*) Couple close roof.
   (*c*) Mansard roof truss.
   (*d*) Couple roof.
5. Give sketches of king-post truss and queen-post truss. Compare the two.

6. Differentiate clearly between (*i*) single roof, (*ii*) double roof, and (*ii*) trussed roofs.

7. Compare steel roof trusses and timber roof trusses.

8. Explain the following :

(*i*) Tiles roofing on pitched roofs.

(*ii*) A.C. sheet roofing.

(*iii*) Mud-phuska roofing.

(*iv*) Slate roofing.

9. Explain any method of providing water proof terracing on R.C.C. roof slab.

10. Explain Jodhpur type lime terracing.

16

Carpentry and Joinery

16.1. INTRODUCTION

The timber, to be used for structural construction (such as door frames, window frames, trusses etc.) is to be dressed, planed, framed and placed in proper position. *Carpentry* is a term applied to that form of wood construction which has to resist stresses due to loads coming on it. Such wood construction members may be permanently subjected to bear the loads (as in the case of wooden lintels, beams, trusses, roofs, floors) or they may be subjected to bear the load temporarily (such as in scaffolding, centering, form work, shoring etc.). Thus, a *Carpenter* constructs *structural timber works,* such as roofs, floors, scaffolding shoring etc. The term *joinery* may be defined as the trade in wood work in which skilled labour is required to render the wooden members capable of framing together. It is the art of preparing internal fittings and finishing of timber. A *joiner*, thus, constructs timber works such as doors, windows, stairs, floor boards, cupboards, furniture etc. Joinery is used for delicate construction, requiring precise workmanship, for enhancing the architectural beauty of timber.

In India, carpentry and joinery are treated as a *single trade.* The word *carpentry* is used to indicate both carpentry and joinery, and the workman who handles the work of carpentry and joinery is called a *carpenter.*

16.2. TECHNICAL TERMS IN CARPENTRY

The following technical terms are commonly used in carpentry.

1. Sawing. It is the art of cutting wood by means of a saw.

2. Shooting. It is the art of dressing of edges of timber pieces so as to make them straight and square with the face.

3. Chamfering. It consists of taking off the edge or corner or arras of a wooden member. The chamfered member has a sloping edge which usually has a slope of 45°. If the angle of chamfer is

other than 45°, it is known as *bevel*. If the chamfer does not continue for full length of a member, it is known as *stopped chamfer*.

4. Planning. It is the process of taking the shaving off wood, with the help of a tool known as a planer. Due to this, timber surfaces are made smooth. Planed or smoothened surface is known as *dressed* or *wrought surface*.

5. Mitring and Scribing. *Mitring* is the process of joining two wooden members at an angle. If one end of moulding is cut to suit the profile of another moulding, it is known as *scribing*.

6. Moulding. It is the process of shaping various units of construction, either by hand or by machine, to produce moulded sections.

7. Rebating. It is the process of cutting a rectangular groove on the edge of a timber piece so as to enable thc edge or tongue of another timber piece to fit in the former.

8. Housing. It is the process of sinking of edge of one piece of timber into the another, by cutting groove across its grains. Housing may be plain, shouldered or dove-tailed.

9. Groove and grooving. Groove is a term used to indicate a recess formed in a timber member. If the groove is made parallel to the grains, it is known *plough grooving,* while if the groove is made across the grains, it is known as *cross grooving*.

10. Nosing.. Nosing is the edge of portion overhanging a vertical surface.

11. Studding. It is the term applied to the fixing of small timber battens to timber walls, to which lathes and boards are to be nailed.

12. Batten. It is a narrow strip of wood which is nailed over joints of boards.

13. Veneering. It is the process of covering of entire or part of exposed surface of timber by means of veneers, for decorative purposes.

14. Wain-Scot. It is the wooden panelling applied on masonry walls for a height of 60 cm from the floor level.

15. Bead. It is rounded or semi-circular moulding provided on the edges or surface of wood.

16.3. PRINCIPLES GOVERNING THE CONSTRUCTION OF JOINTS

Joints play the most important role in timber construction since they provide structural stability, improve aesthetic appearance, and facilitate the construction. However, joints are the weakest parts of a timber structure. Hence the following principles, based on the recommendations by Prof. Rankine, should be followed in the con-

struction of joints :

1. The joint should be cut and placed in such a way that it weakens the connecting members to the minimum.

2. Each abutting surface of joint should be, as far as possible, normal to the line of pressure coming upon the joint.

3. Each abutting surface of a joint should be designed for the maximum compressive stress likely to come upon it.

4. The surface of a joint should be formed and fitted accurately so that there is even distribution of pressure.

5. The fastenings should be proportioned in such a way that they possess equal strength in relation to the members which they connect.

6. Fastenings should be placed and designed in such a manner as to avoid failure of joint by shear or crushing.

7. The joint should be simple as far as possible. Complicated joints are difficult to construct, take more time, easily affected due to shrinkage of timber and get easily attacked by vermins due to the presence of many surfaces and angles.

16.4. CLASSIFICATION OF JOINTS

Various types of joints used in wood work may be classified under the following heads :

1. Lengthening joints.
2. Widening joints or side joints.
3. Bearing joints.
4. Framing joints.
5. Angle or corner joints.
6. Oblique shouldered joints.

16.5. LENGTHENING JOINTS

These are also known as *spliced or longitudinal joints.* These joints are used to increase the length of wood members, such as ties, struts, etc. The method of lengthening depends upon the situation of the member in a framed structure, where such joints are commonly required. Lengthening joints are of the following types :

1. Lapped joints.
2. Fished joints.
3. Scarfed or spliced joints.
4. Tabled joints.

1. Lapped joints

This is the simpiest type of joint, formed by placing the two ends of the members one over the other for a short distance and binding them together by means of wrought iron straps and bolts. If the member carries tensile stress, it is essential to provide bolts passing through both the pieces (Fig. 16.1).

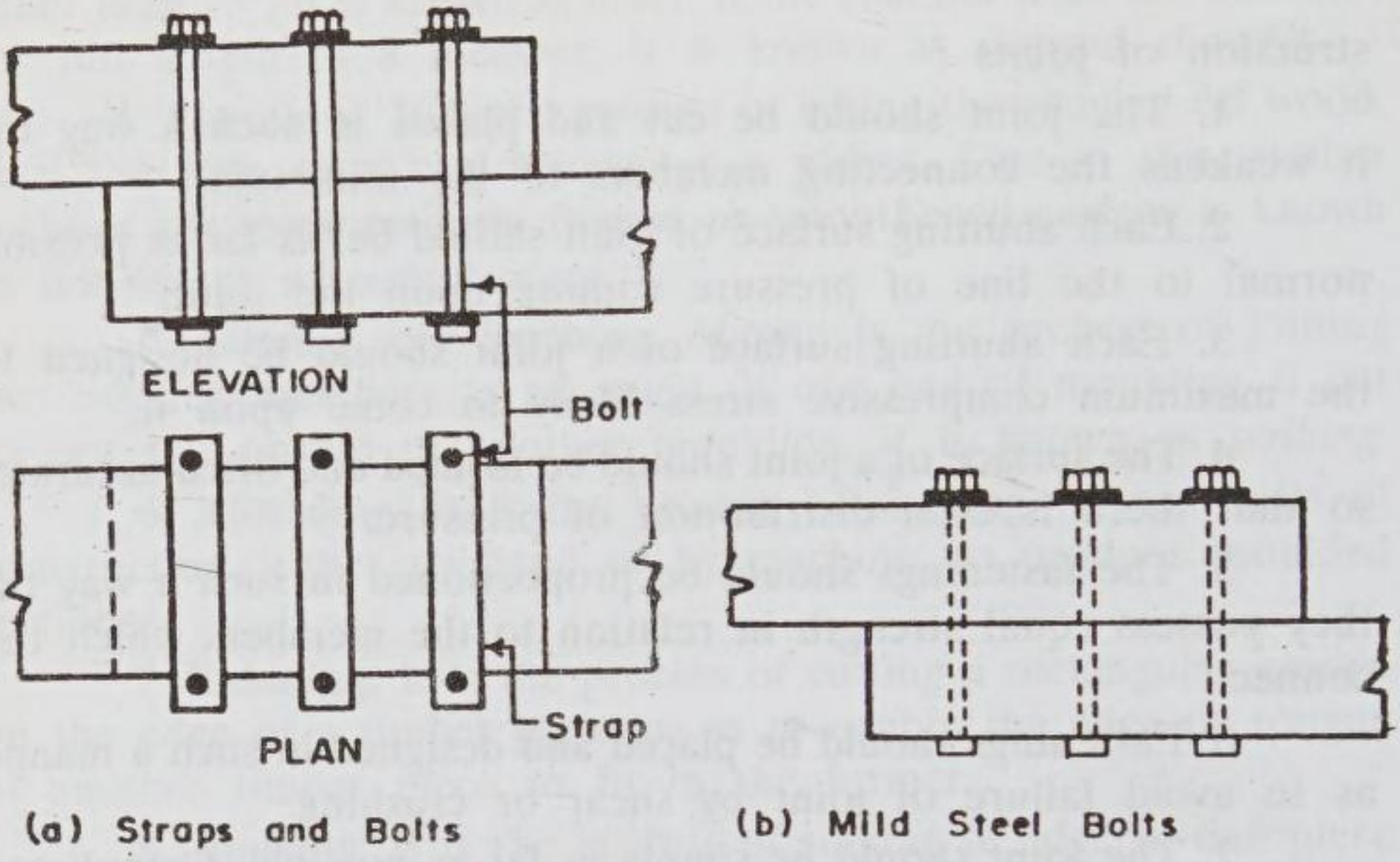


FIG. 16.1. LAPPED JOINT.

2. Fished Joint

In this joint, the ends of the two members are cut square and placed touching each other (or butted). They are then jointed together placing wooden or iron fish plates on opposite faces and securing these by passing bolts through them, as shown in Fig. 16.2(*a*). The ends of fish plates are slightly bent and then pressed into the

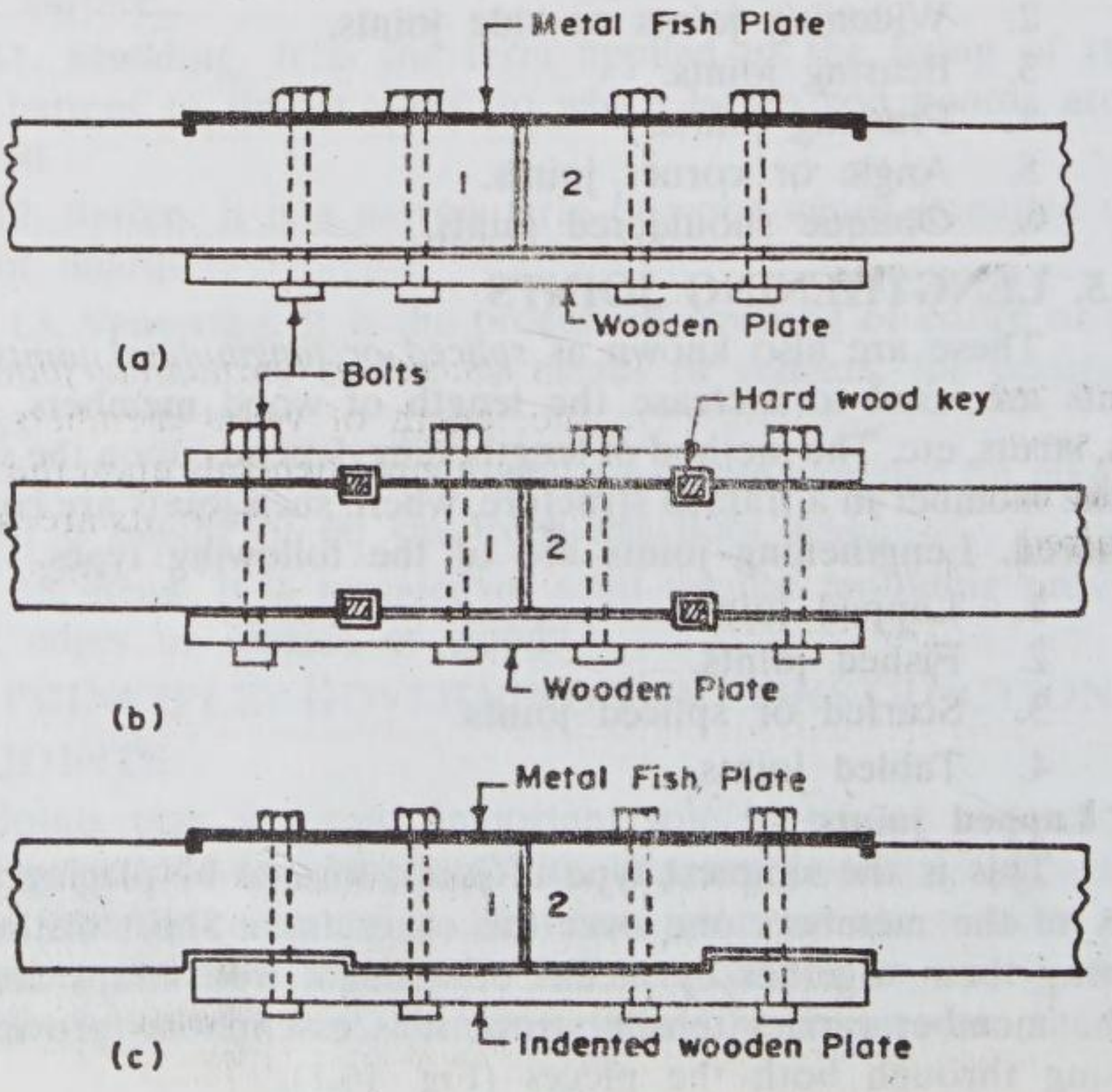


FIG. 16.2. FISHED JOINTS.

members. Figs. 16.2 (*b*) and (*c*) show other forms in which the joint is further strengthened by keys or indented fish plates.

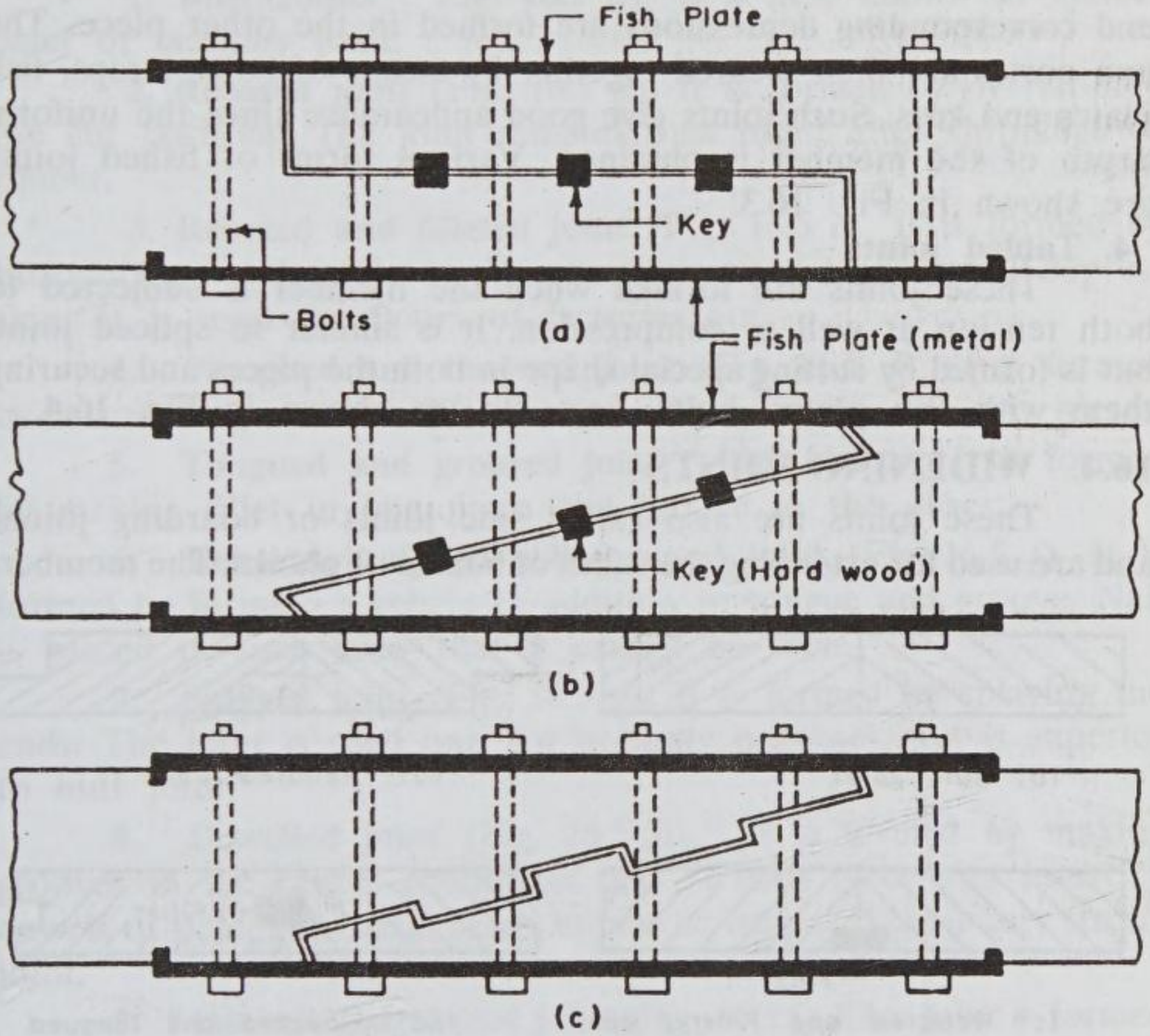


FIG. 16.3. SCARFED OR SPLICED JOINTS.

FIG. 16.4. TABLED JOINTS.

3. Scarfed or spliced joint

In this joint, projections are made in the end of one piece and corresponding depressions are formed in the other piece. The two pieces are then secured together by means of bolts, straps, fish plates and keys. Such joints give good appearance since the uniform depth of the member is obtained. Various forms of fished joints are shown in Fig. 16.3.

4. Tabled joints

These joints are formed when the member is subjected to both tension as well as compression. It is similar to spliced joint, but is formed by cutting special shape in both the pieces and securing them with fish plates, bolts, keys etc., as shown in Fig. 16.4.

16.4. WIDENING JOINTS

These joints are also cailed side joints or boarding joints, and are used for extending the width of boards or planks. The members

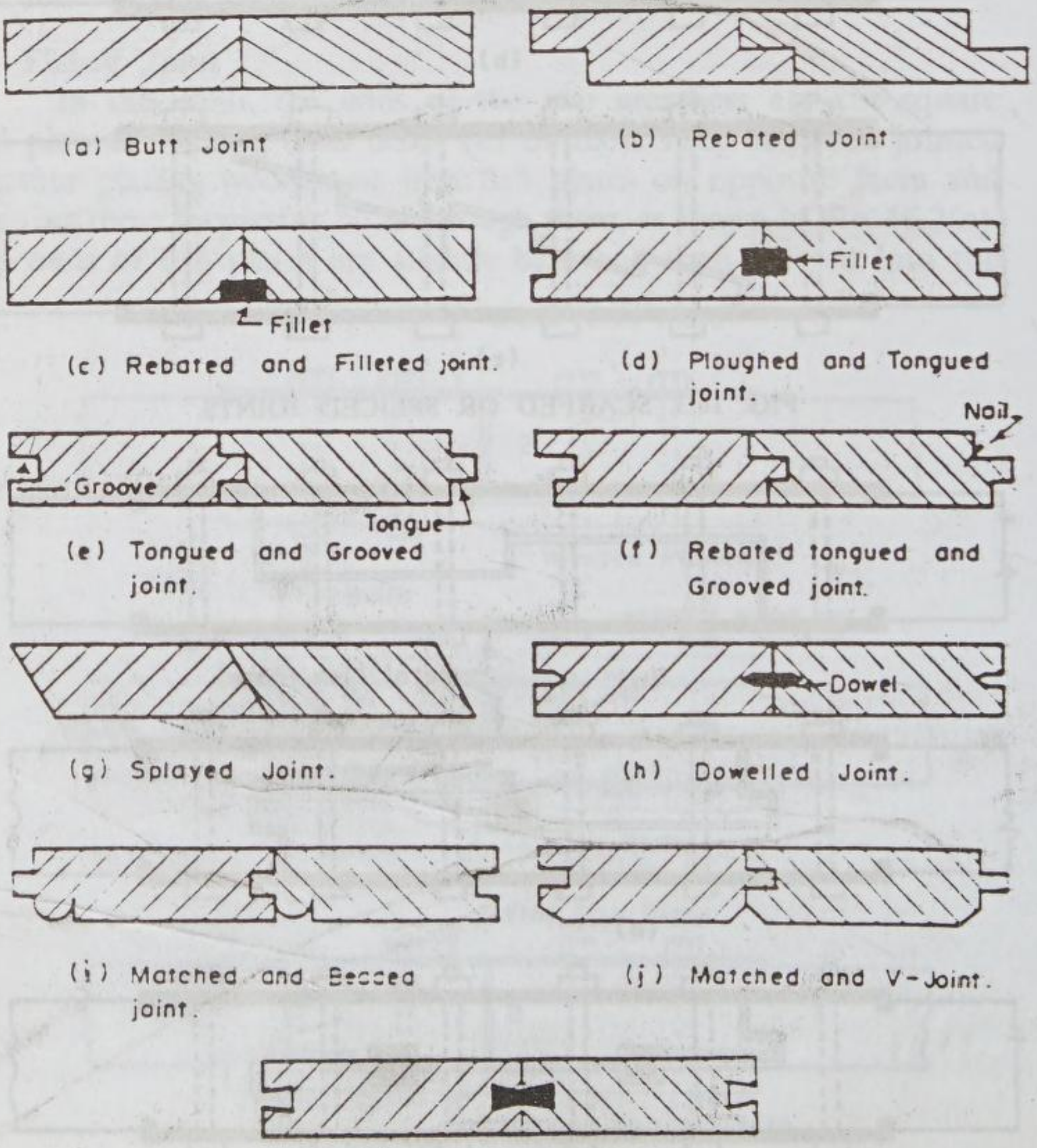


FIG. 16.5. WIDENING JOINTS.

are placed edge to edge. These are used for wooden doors, floors etc. They are of the followin types :

1. Butt Joints (Fig. 16.5 *a*). It is also known as *square, plain* or *ordinary* joint.

2. Rebated joint (Fig. 16.5 *b*). It is formed by overlapping the cut portions. The joint remains dust proof after shrinkage of timber.

3. Rebated and filleted joint (Fig. 16.5 *c*). It is formed by introducing wooden fillet in the rebated portions, having small depression. It is used for floors of factories etc.

4. Ploughed and tongued joint (Fig. 16.5 *d*). It is formed by introducing wooden fillet in the grooves cut in the two pieces.

5. Tongued and grooved joint (Fig. 16.5 *e*). It is formed by making fillet in one piece and groove in the other.

6. Rebated, tongued and grooved joint (Fig 16.5 *f*). It is formed by forming a rebate in addition to tongue and groove. Nail is placed in such away that it cannot be seen.

7. Splayed joint (Fig. 16.5 *g*). It is formed by splaying the ends. The joint is used only for ordinary purposes, but is superior to butt joint.

8. Dowelled joint (Fig. 16.5 *h*). It is formed by making grooves in the centre portion of end of each piece and inserting dowels of slate, gun-metal brass, bronze or copper. This is very strong joint.

9. Matched and beaded joint (Fig. 16.5 *i*). This joint is formed by tongue and groove arrangement, and has special moulding on one side to give good appearance.

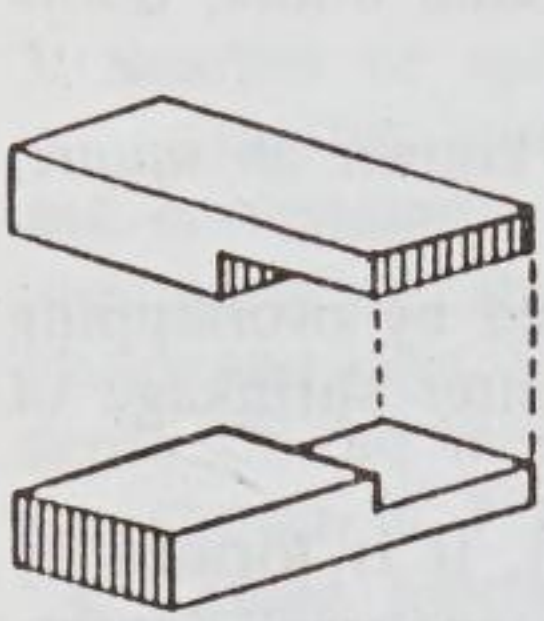
10. Matched and V-jointed joint (Fig. 16.5 *j*). This is similar to the beaded joint, except that it is chamfered in the shape of V.

11. Dovetailed Joint (Fig. 16.5 *k*). It is formed by providing key of dovetail shape to fit in the corresponding grooves in the connecting members.

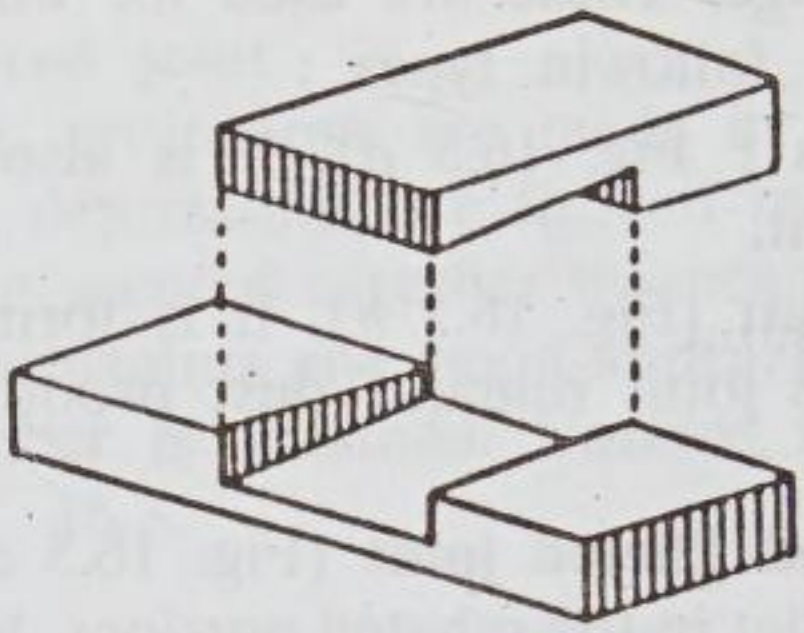
16.7. BEARING JOINTS

Bearing joints are provided when two members meet at right angles, to give sufficient strength to the functions. Bearing joints are of the following types :

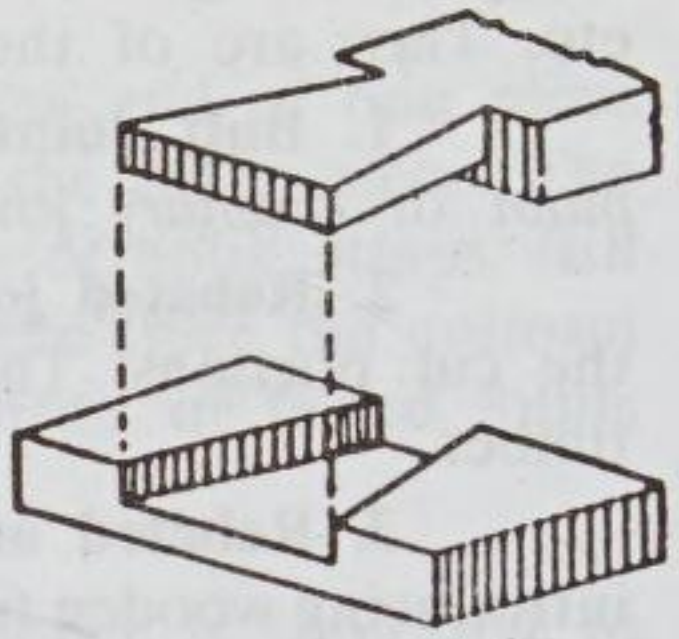
1. Halved joints. These joints are formed by cutting through half the depth of each member meeting at right angles, so that top surfaces of both the members flush. Fig. 16.6 (*a*) shows angle halved joints. Fig. 16.6 (*b*) shows bevel halved joint. Fig 16.6 (*c*) shows dovetailed halved joint. Figs. 16.6 (*d*) and (*e*) show respectively longitudinal halved and tee halved joints.



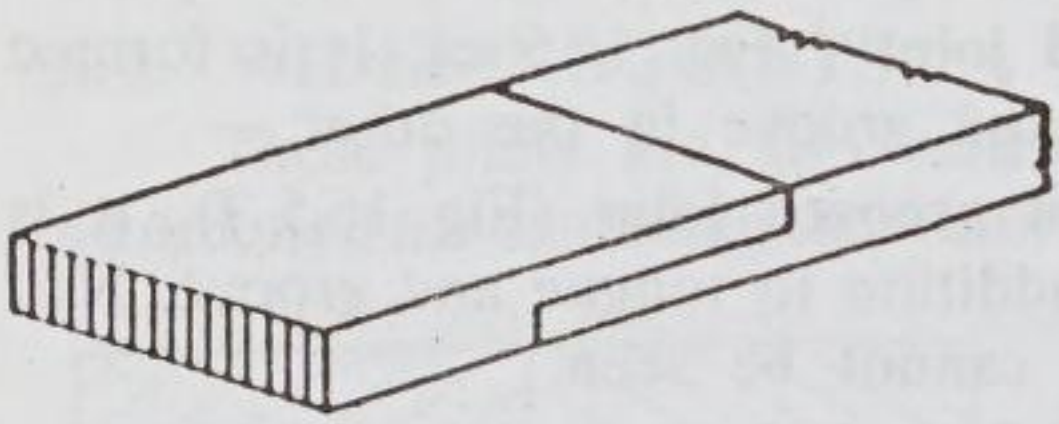
(a) Angle Halved Joint



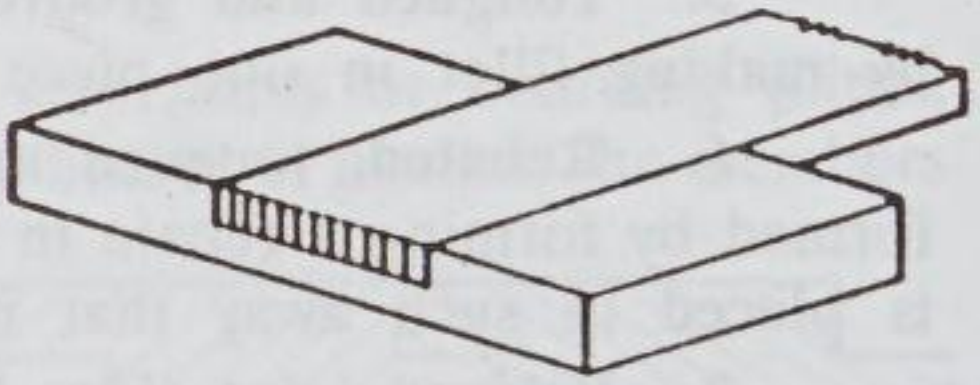
(b) Bevel Halved Joint



(c) Dovetail Halved Joint



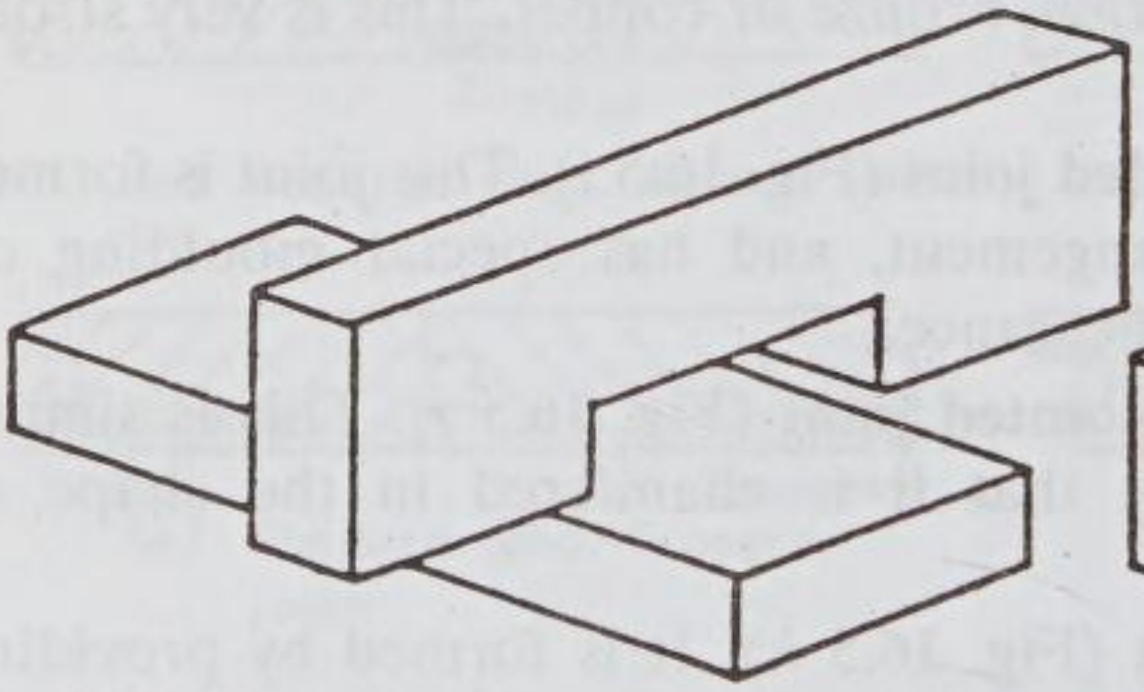
(d) Longitudinal Halved Joint



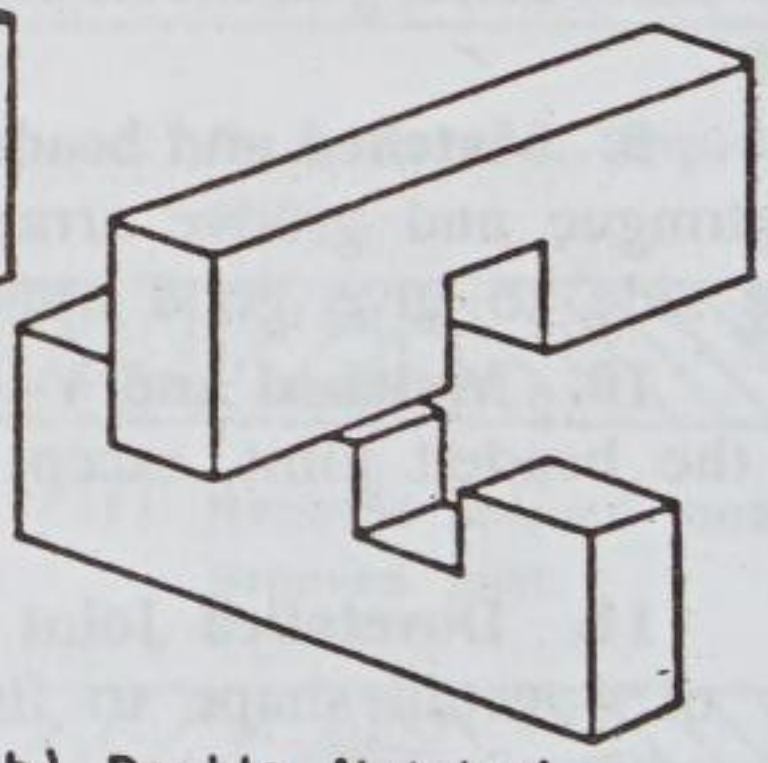
(e) Tee Halved Joint

FIG. 16.6.

2. Notched Joint. It is formed by forming notch in one or both the members to be connected (Figs. 16.7 *a*, *b*).



(a) Single Notched



(b) Double Notched

FIG. 16.7. NOTCHED JOINT.

3. Cogged joint (Fig. 16.8). This joint is formed by cutting small notch in the beam or timber member and providing notches on the lower member with a projection in the centre. The projection is known as *cog*. The upper portion, in which only small notch has been formed, retains its strength.

4. Housed joint (Fig. 16.9 *a*). It is formed by fitting the entire tickness of the end of one member for a short distance into another piece. It is used in stairs in which the ends of risers and treads are housed in the strings.

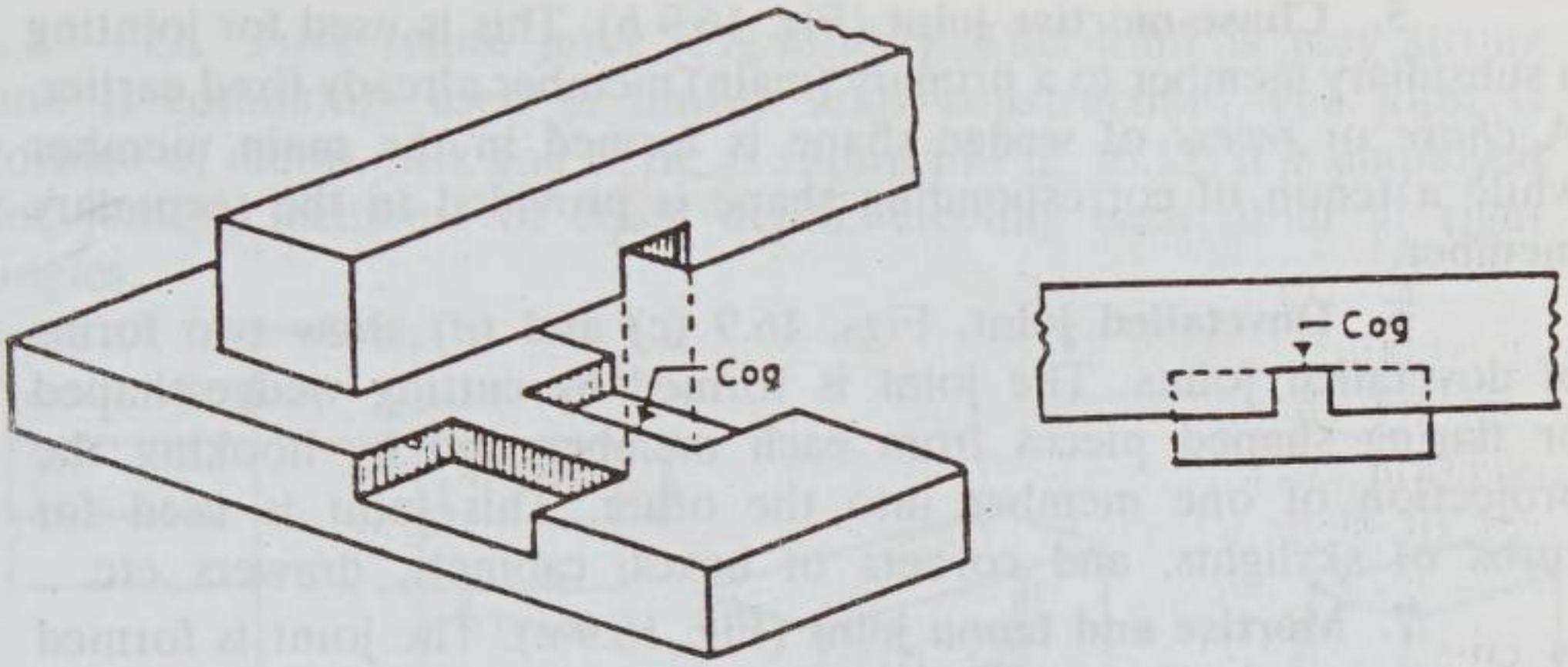
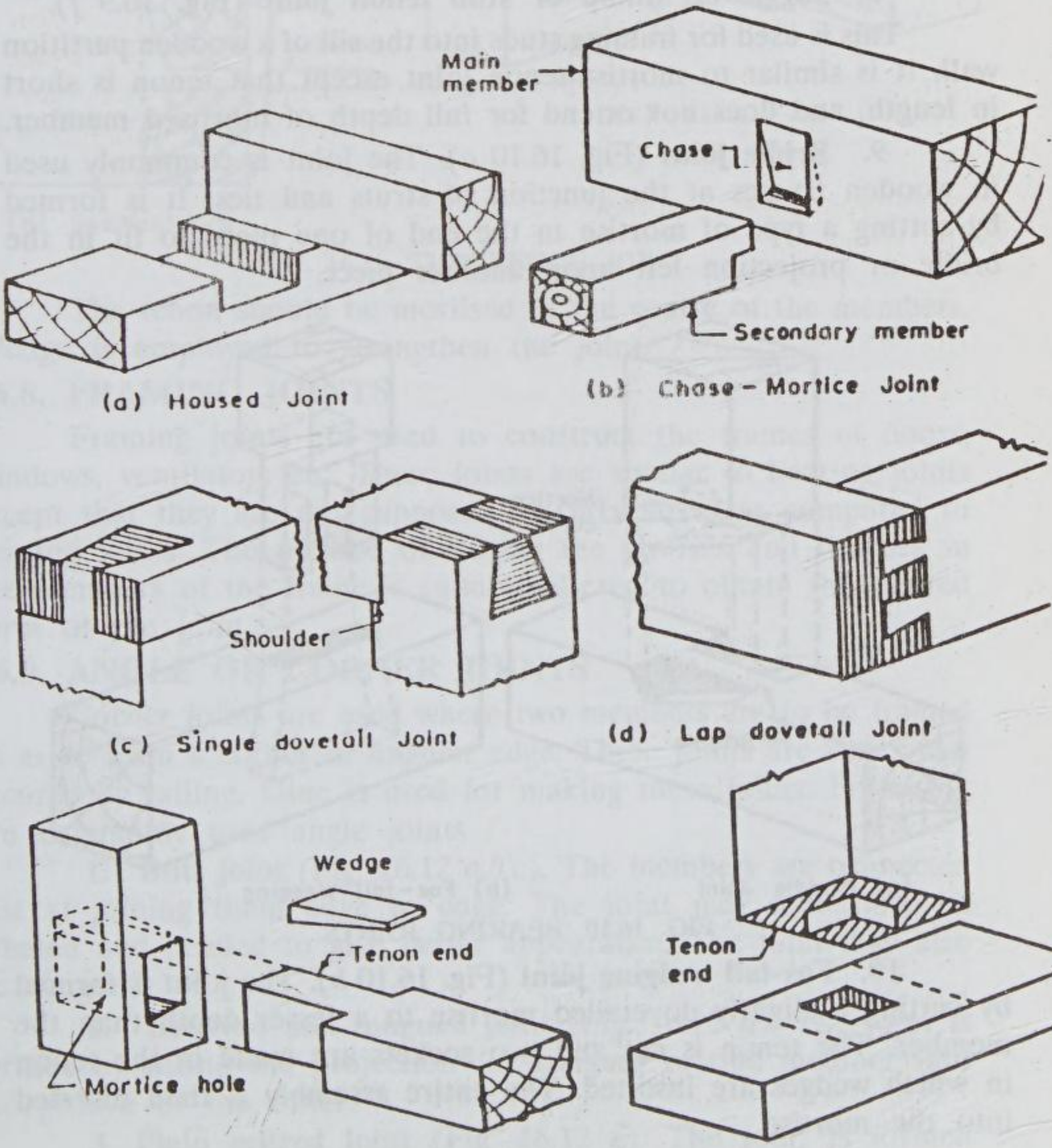


FIG. 16.8. COGGED JOINT



(a) Housed Joint

(b) Chase – Mortice Joint

(c) Single dovetail Joint

(d) Lap dovetail Joint

(e) Mortice and Tenon Joint

(f) Stub Tenon Joint

FIG. 16.9. BEARING JOINTS.

5. Chase-mortise joint (Fig. 16.9 *b*). This is used for jointing a subsidiary member to a primary (main) member already fixed earlier. A *chase* or *recess* of wedge shape is formed in the main member while a tenon of corresponding shape is provided in the secondary member.

6. Dovetailed joint. Figs. 16.9 (*c*) and (*d*) show two forms of dovetailed joints. The joint is formed by cutting wedge-shaped or flaring shaped pieces from each member and by hooking the projection of one member into the other. This joint is used for curbs of skylights, and corners of boxes, cabinets, drawers etc.

7. Mortise and tenon joint (Fig. 16.9 *e*). The joint is formed by cutting projection, known as tongue or tenon, in one member which fits into slot, called mortise, cut into the other member.

8. Joggle or stump or stub tenon joint (Fig. 16.9 *f*).

This is used for framing studs into the sill of a wooden partition wall. It is similar to mortise-tenon joint except that tenon is short in length, and does not extend for full depth of mortised member.

9. Bridle joint (Fig. 16.10 *a*). The joint is commonly used in wooden trusses at the junction of struts and ties. It is formed by cutting a type of mortise in the end of one piece to fit in the bridle or projection left upon another piece.

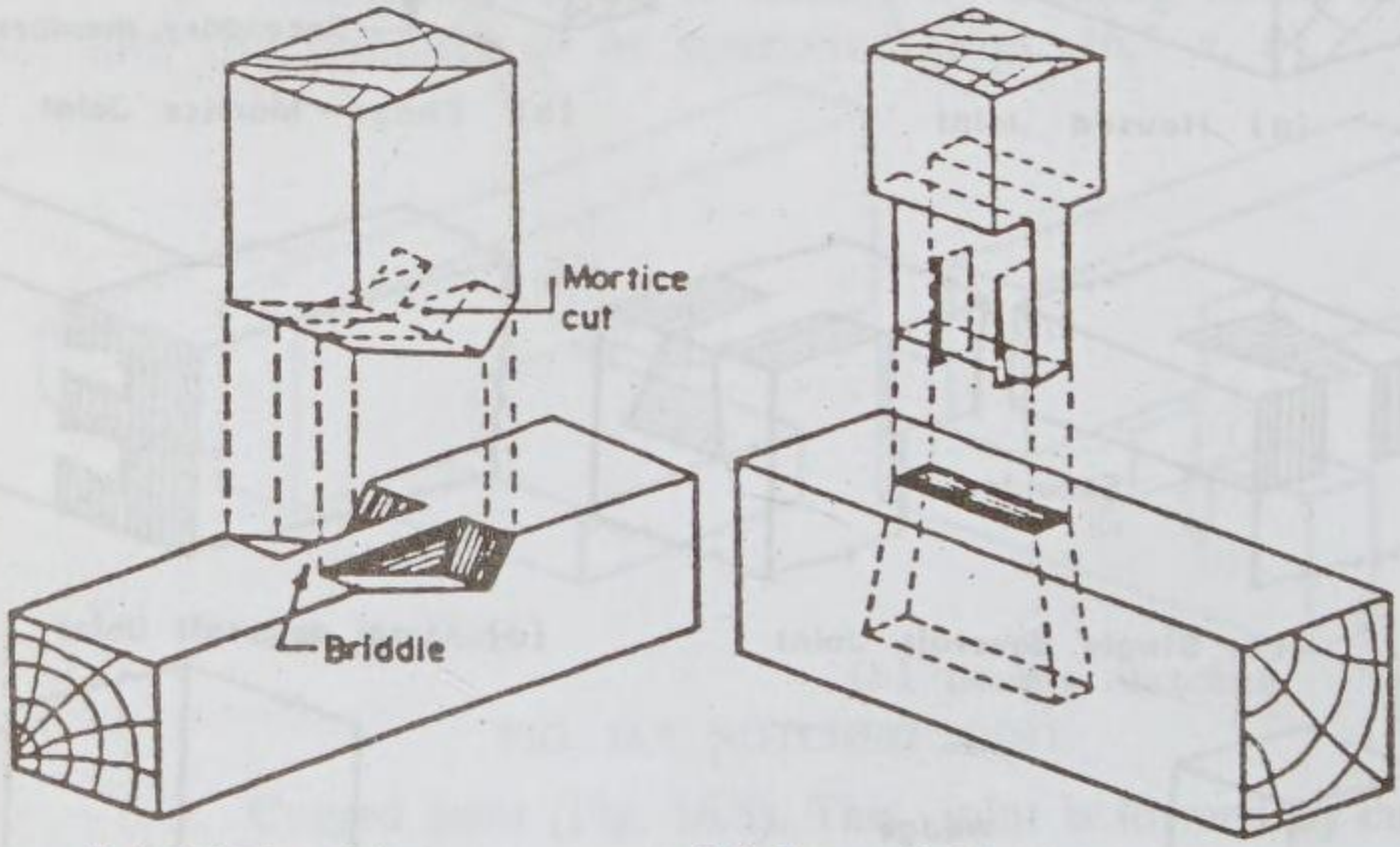


FIG. 16.10. BEARING JOINTS.

10. Fox-tail wedging joint (Fig. 16.10 *b*). The joint is formed by cutting a slightly dovetailed mortise to a lesser depth than the member. The tenon is cut and two sockets are made in the tenon in which wedges are inserted. The entire assembly is then inserted into the mortise.

11. Tusk tenon joint (Fig. 16.11). This joint is very strong and is commonly used in timber floor construction. The joint is formed of tenon, tusk and horn, as shown in Fig. 16.12. It is employed for joining members of equal depth, meeting each other at right angles.

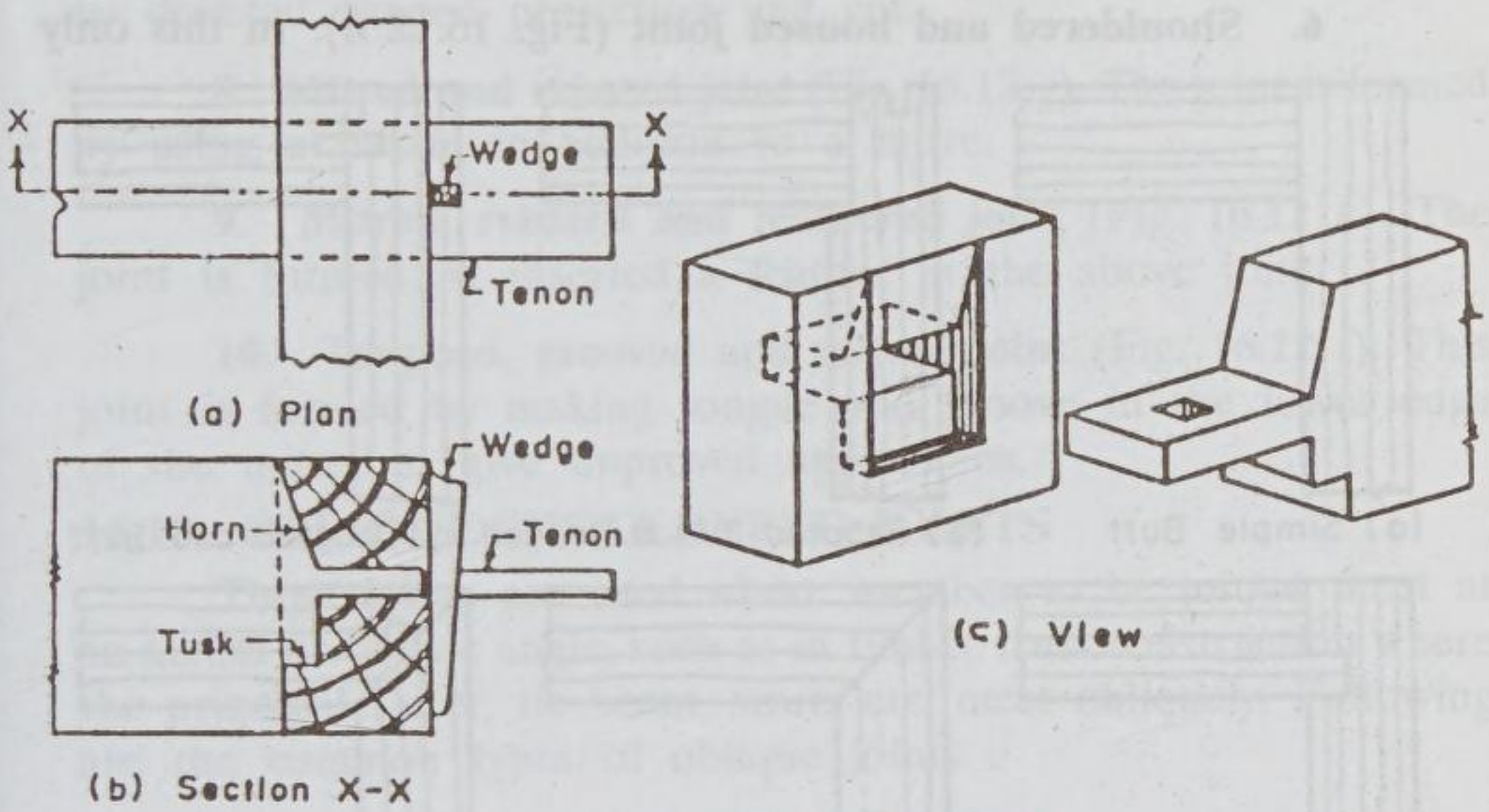


FIG. 16.11. TUSK-TENON JOINT.

The tenon should be mortised in the centre of the members. Wedge is employed to strengthen the joint.

16.8. FRAMING JOINTS

Framing joints are used to construct the frames of doors, windows, ventilators etc. These joints are similar to bearing joints except that they are not supposed to carry stress as compared to bearing joints. The method of cutting the grooves and tongues in the members of the frame is suitably altered to obtain the desired form of the joint.

16.9. ANGLE OR CORNER JOINTS

Corner joints are used where two members are to be framed so as to form a corner or angular edge. These joints are very often secured by railing. Glue is used for making these joints. Following are commonly used angle joints :

1. **Butt joint** (Fig. 16.12 *a,b,c*). The members are connected just at joining them edge to edge. The joint may sometimes be rebated and beaded to give better appearance. The joint may also be tongued.

2. **Grooved and tongued joint** (Fig. 16.12 *d*). The joint is formed by fitting the projection (or tongue) of one member into the groove of the other.

3. **Plain mitred joint** (Fig. 16.12 *e*). The joint is formed by cutting the edges of both the members at an angle.

4. Mitred and feathered joint (Fig. 16.12*f*). In this, an additional wooden member is inserted in the middle of the mitre joint.

5. Housed Joint (Fig. 16.12 *g*). The joint is formed by fitting on member completely into the depression of the other.

6. Shouldered and housed joint (Fig. 16.12 *h*). In this only

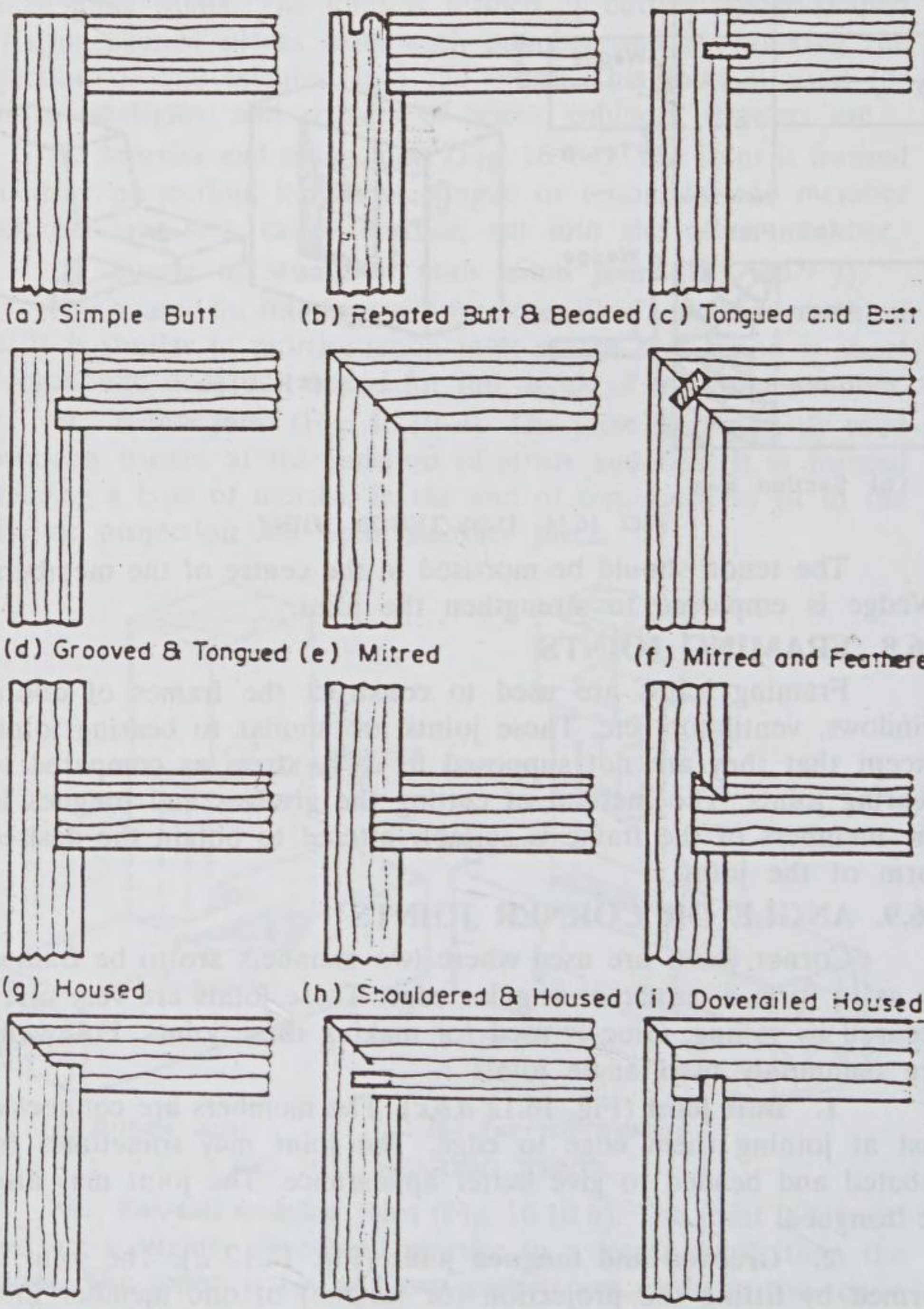


FIG. 16.12. ANGLE JOINTS.

a part of one member fits into the corresponding depression of the other.

7. Dovetailed housed joint (Fig. 16.12 *i*). This is a special type of housed joint in which one member is housed into the other by dovetail shaped projection and cut.

8. Mitred and rebated joint (Fig. 16.12. *j*). The joint is formed by using a rebate in addition to a mitre.

9. Mitred, rebated and feathered joint (Fig. 16.12 *k*). The joint is formed by inserted a feather in the above joint.

10. Tongued, grooved and mitred joint (Fig. 16.12 *l*). This joint is formed by making tongue and groove in the lower edge of the mitre, to give improved appearance.

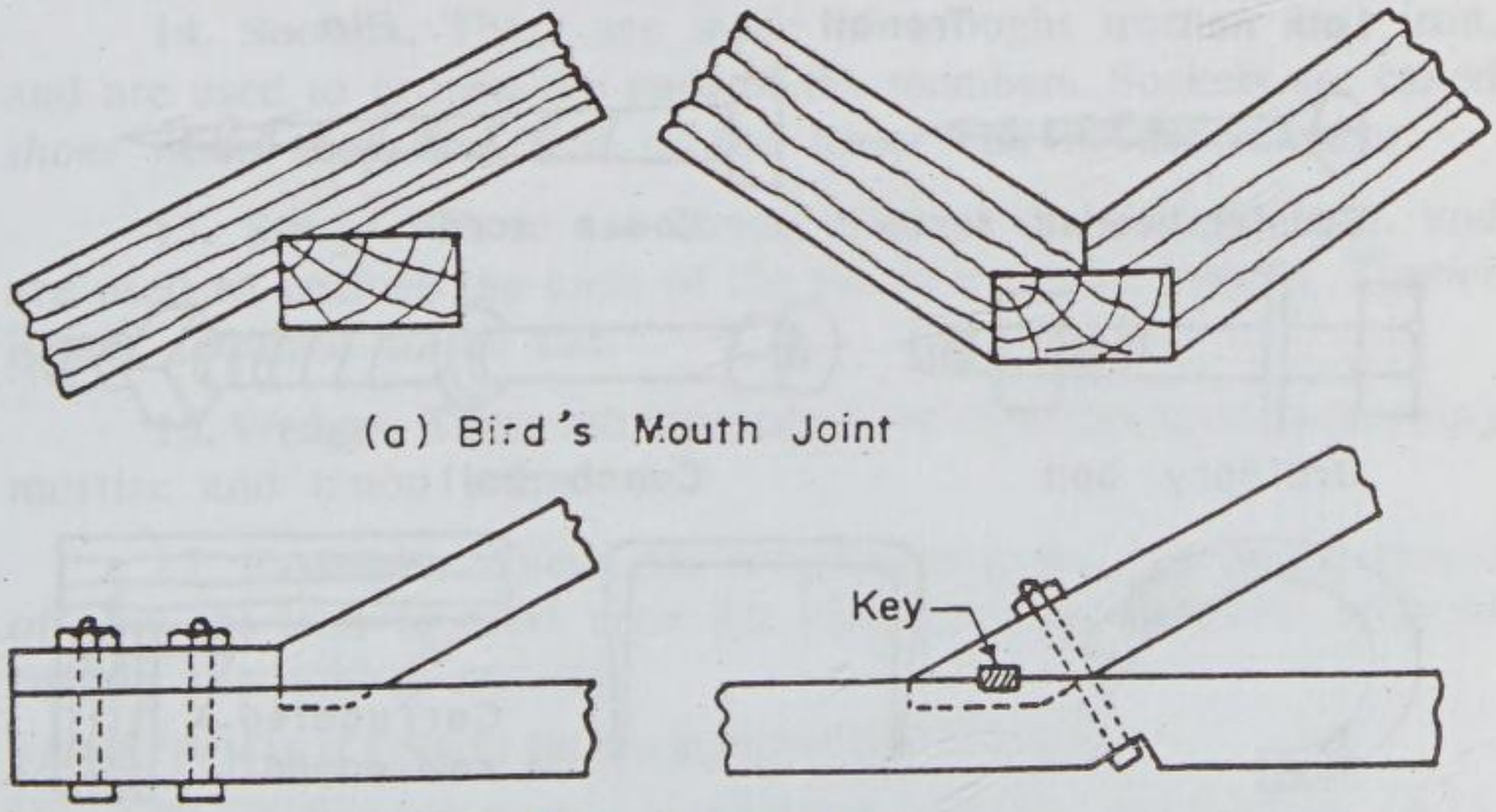
16.10. OBLIQUE SHOULDERED JOINTS

These joints are used where member to be joined meet at an actute or obtuse angle, such as in timber truss construction where the principal rafter, tie beam, struts etc, meet obliquely. Following are the common types of oblique joints :

1. **Briddle joint**
2. **Mitred joint**
3. **Dovetailed halved joint**

These joints (shown in Figs. 16.10 *a*, 16.12 *e* and 16.6 *c* respectively) are similar to those discussed earlier, except that the members meet at an angle, other than a right angle.

4. **Birds mouth joint** (Fig. 16.13 *a*). This joint is formed by



(a) Bird's Mouth Joint

(b) Oblique Tenon Joint

FIG. 16.13. OBLIQUE TENON JOINTS.

cutting an angular notch (called *bird's mouth*) in the main member, to which the other member is partly inserted and fitted.

5. Oblique tenon joint (Fig. 16.13 *a*). This is used for connecting a horizontal member to an inclined member, both the members being of bigger size. The tenon of inclined member is oblique, which fits into the corresponding mortise hole of the horizontal member. The joint is further strengthened by bolts, keys, straps etc.

16.11. FASTENINGS

Timber joints are secured in position with the help of following commonly used fastenings (Fig. 16.14).

1. Wire nails. These are circular or oval in shape, made of wrought iron or steel.

2. Cut nails. These are trapezoidal in section, and are smaller in length.

3. Floor brads. These are tapering nails of rectangular section, with head at one end, and are used for securing floor boards.

4. Lath nails. It is in the form of iron clout, square and

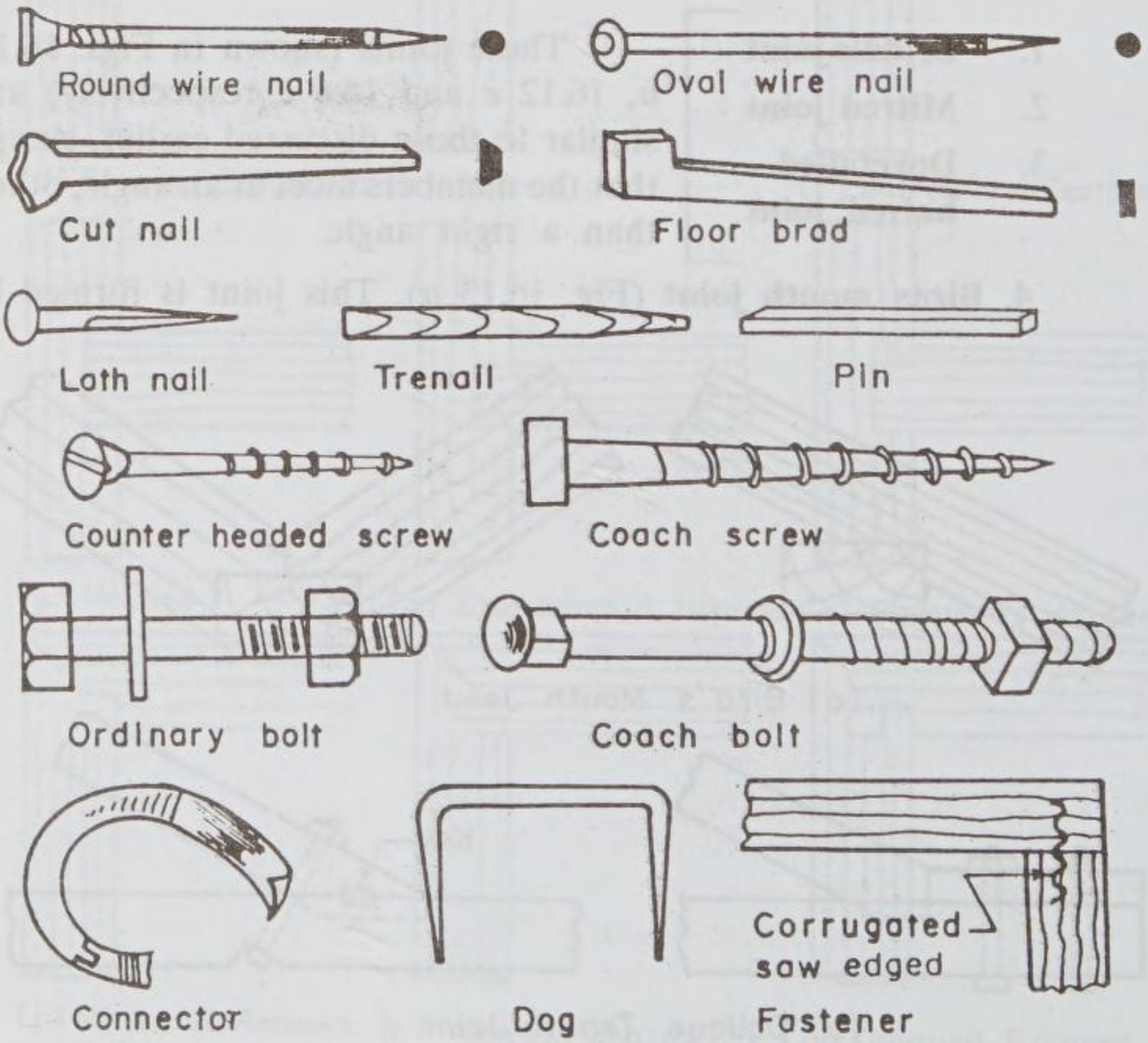


FIG. 16.14. FASTENINGS.

tapering, with rough sides.

5. Trenail. It is a nail or pin of hard wood.

6. Pins. These are small wooden pieces used for securing joints of door and window shutters.

7. Screws. These are used where (*i*) work is temporary, (*ii*) flexible joint is required, (*iii*) driving nail is likely to split the joint, and (*iv*) joint is subjected to vibrations. They make the joint stronger because of their greater holding power. These may be round-headed or counter-sunk.

8. Coach screw. It has a square head which is turned by a spanner.

9. Bolts. These are used for large size members. Washers are used with nuts, to prevent injury to timber. These are used for joining members carrying tensile stresses.

10. Spikes. These are large nails of 10 to 15 cm length, used to secure heavy members.

11. Connectors. These are metal rings or corrugated sheet pieces which are driven into the members after abutting them.

12. Dig. A dog is a U-shaped wrought-iron fastening with pointed ends, which is driven to connect the members. It is used for temporary structures.

13. Dowels. These are small wooden pieces which are driven in the members to keep their faces in one plane.

14. Sockets. These are made of wrought iron or cast iron, and are used to protect the ends of the members. Sockets are called *shoes* when they are fixed to the lower end of the member.

15. Straps. These are bands of steel or wrought iron, and are used to enclose the ends of the members to be jointed. Timber is not required to be cut.

16. Wedges. These are tapered pieces of wood, used in securing mortise and tenon joints.

17. Fasteners. These are rough corrugated saw edged pieces of wrought iron or steel used for strengthening the joint, without cutting the timber sections.

16.12. TOOLS USED IN CARPENTRY WORK

The following tools are used for carpentry work (Figs. 16.15, 16.16 and 16.17) :

1. Marking tools. These are used for marking lines on wood.

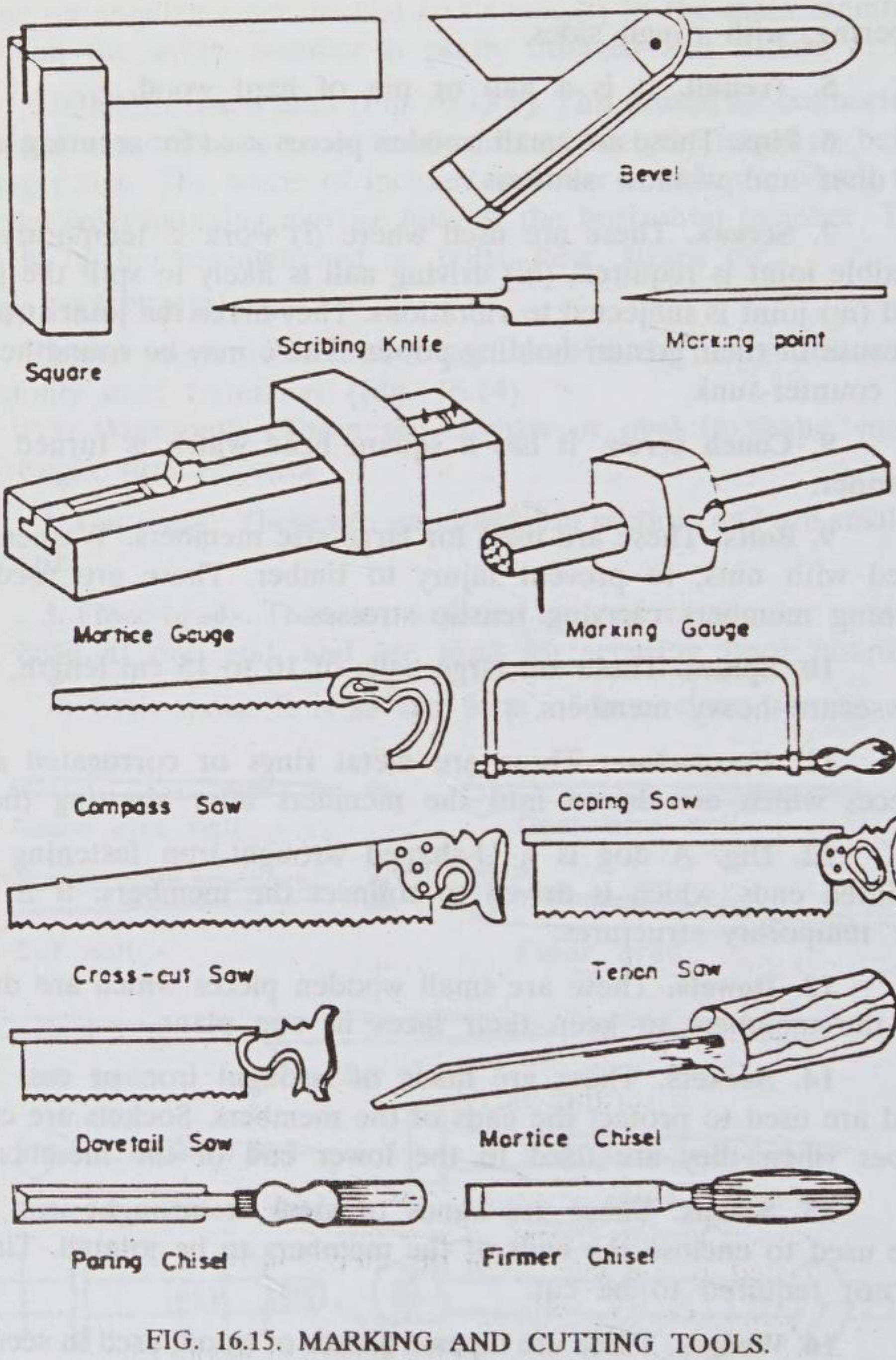


FIG. 16.15. MARKING AND CUTTING TOOLS.

(*i*) *Square.* To set right angles.

(*ii*) *Bevel.* To set angles other than a right angle.

(*iii*) *Marking gauge and mortise gauge..* Used for marking lines parallel to the edges.

(*iv*) *Marking point and scribing knife.* To mark points and lines on wood.

2. Cutting tools :

(*i*) *Compass saw.* Used for cutting wood.

(*ii*) *Coping saw*
(*iii*) *Cross–cut saw*
(*iv*) *Dovetail saw*
] Used for cutting timber members.

(*v*) *Tenon saw*
(*vi*) *Firmer chisel*
(*vii*) *Mortise chisel*
(*viii*) *Paring chisel*
] Used for cutting and shaping joints.

3. Boring tools. Used for driving holes in timber members.

(*i*) *Ratchet brace.* Cutting bit is attached to its lower end. The bit is rotated with the help of brace handle.

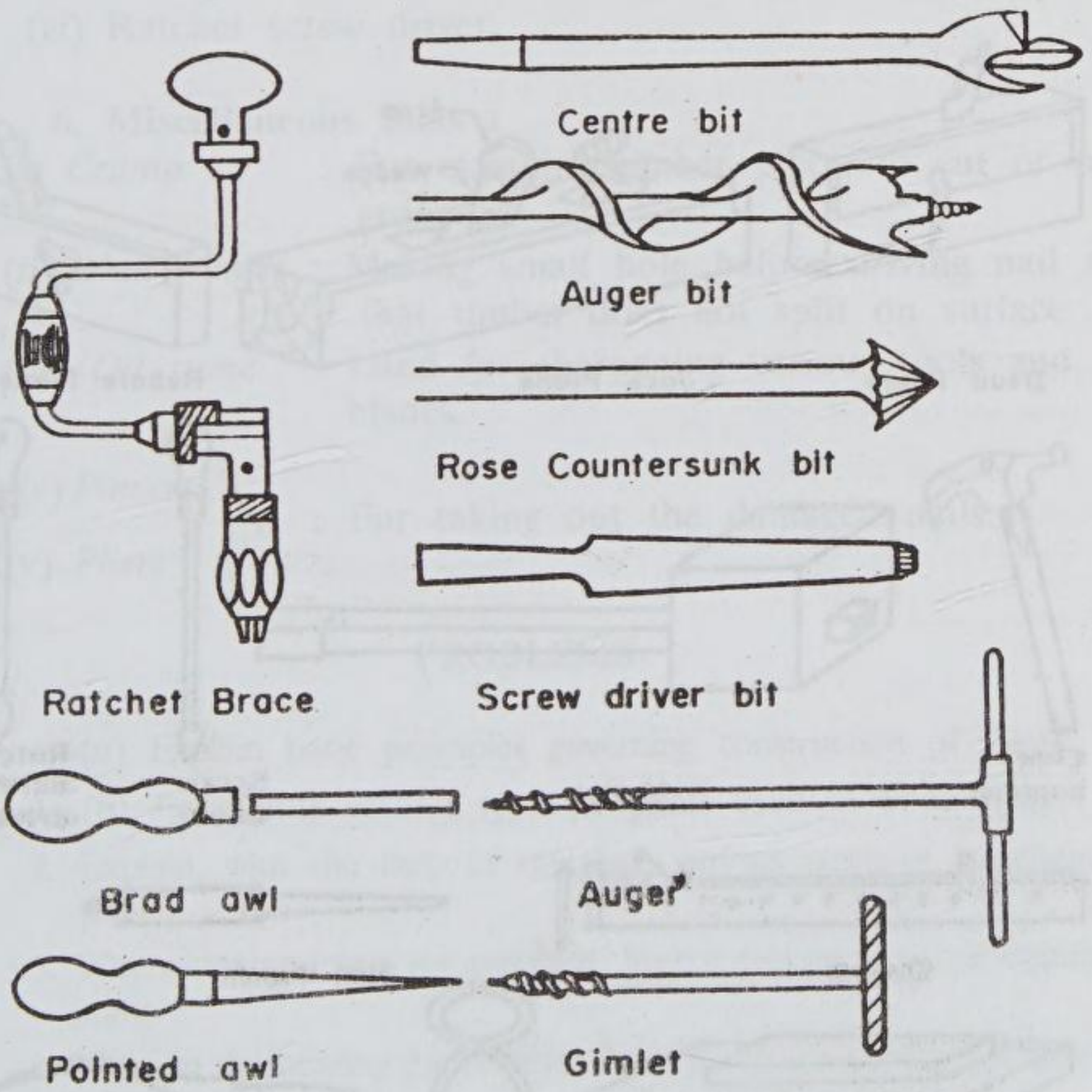


FIG. 16.16. BORING TOOLS.

(*ii*) *Centre bit*
(*iii*) *Auger bit*
(*iv*) *Rose counter–sunk bit*
(*v*) *Screw drive but*

] Used for boring holes of different size and shapes.

(*vi*) *Brad awl*
(*vii*) *Pointed awl*

] These have sharpened and pointed ends with the help of which small and fine holes can be made.

(*viii*) *Gimlet* : It has screwed end with the help of which small holes can be bored.

(*ix*) *Auger* : Used for deep boring.

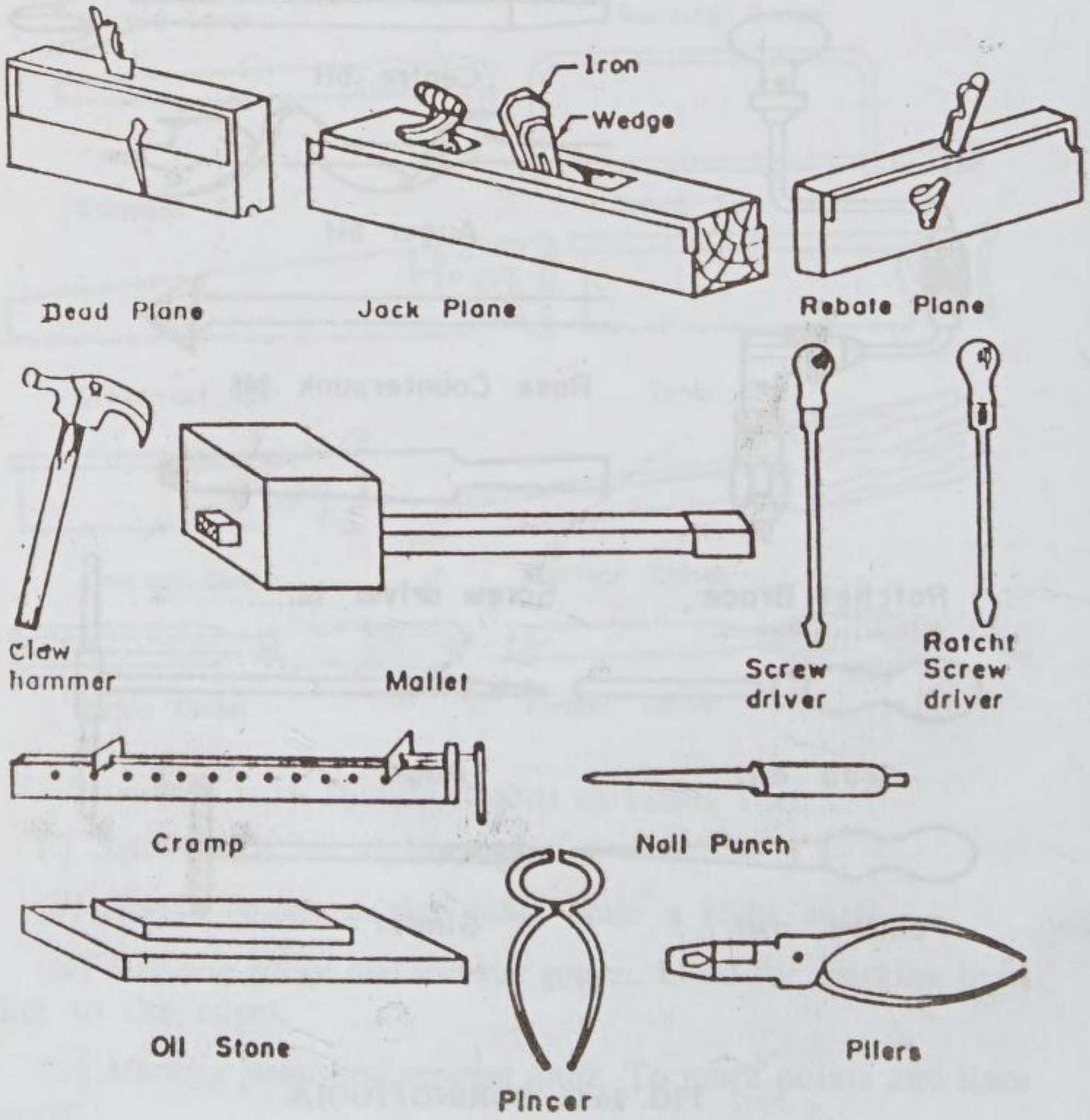


FIG. 16.17. PLANES, HAMMERS AND OTHER TOOLS.

4. Planing tools :

(*i*) *Bead plane*
(*ii*) *Jack plane*
(*iii*) *Rebate plane*

] Used for planing surfaces and for cutting small mouldings along the edges.

5. Hammers and screw drivers Used for driving nails and screws, and other fastenings.

(*i*) Claw hammer.
(*ii*) Mallet hammer.
(*iii*) Spall hammer.
(*iv*) Waller's hammer.
(*v*) Screw drivers.
(*vi*) Ratchet screw driver.

6. Miscellaneous tools :

(*i*) *Cramp* : For clamping timber piece, to cut or make groove.

(*ii*) *Nail Punch* : Making small hole before driving nail so that timber does not split on surface.

(*iii*) *Oil stone* : Used for sharpening various tools and blades.

(*iv*) *Pincers*
(*v*) *Pliers*

] : For taking out the damaged nails.

PROBLEMS

1. (*a*) Explain basic principles governing construction of joints.
   (*b*) Enumerate various types of joints used in wood work.
2. Explain, with the help of sketches, various types of lengthening joints.
3. Why widening joints are essential. Sketch various types of widening joints.
4. How do the bearing joints differ from other joints ? Sketch various types of halved joints.
5. Explain the following types of joints :
   (*i*) Cogged joint
   (*ii*) Chase mortise joint
   (*iii*) Dovetail joint
   (*iv*) Tenon and mortise joint
   (*v*) Briddle joint.

6. Write short notes on the following :
   (*a*) Fox tail wedge joint
   (*b*) Briddle joint
   (*c*) Tusk-tenon joint
   (*d*) Bird's mouth joint
7. Explain, with the help of sketches, various types of corners joints.
8. Write a note on fastenings used in wood work.
9. Enumerate various tools used in wood work.

17

Doors and Windows

17.1. INTRODUCTION

A *door* may be defined as an openable barrier secured in a wall opening. A door is provided to give as access to the inside of a room of a building. It serves as a connecting link between the various internal portions of a building. Basically, a door consists of two parts : (*i*) door frame, and (*ii*) door shutter. The door shutter is held in position by the door frame which in turn is fixed in the opening of the wall by means of hold-fasts etc.

A *window* is also a vented barrier secured in a wall opening. The function of the window is to admit light and air to the building and to give a view to the outside. Windows must also provide insulation against heat loss, and in some cases, against sound. Some windows are also required to give a measure of resistance to fire. A window also consists of two parts : (*i*) window frame, secured to the wall opening with the help of hold fasts, and (*ii*) window shutters held in position by the window frame.

17.2. LOCATION OF DOORS AND WINDOWS

The following points should be kept in view while locating doors and windows :

1. The number of doors in a room should be kept minimum since larger number of doors cause obstruction, and consume more area in circulation.

2. The location of a door should meet functional requirements of a room. It should not be located in the centre of the length of a wall. A door should preferably be located near the corner of a room—nearly 20 cm away from the corner.

3. If there are two doors in a room, the doors should preferably be located in opposite walls, facing each other, so as to provide good ventilation and free-air circulation in the rooms.

4. The size and number of windows should be decided on

the basis if important factors such as distribution of light, control of ventilation, and privacy of the occupants.

5. The location of a window should also meet the functional requirements of the room, such as interior decoration, arrangement of furniture etc.

6. A window should be located in opposite wall, facing a door or another window, so that cross-ventilation is achieved.

7. From the point of view of fresh air, a window should be located on the northern side of a room.

8. From the point of view of fresh air, a window should be located in the prevalent direction of wind.

9. The sill of a window should be located about 70 to 80 cm about floor level of the room.

17.3. DEFINITION OF TECHNICAL TERMS

Figs. 17.1 and 17.2 show respectively a door and a window. The following are the technical terms applied to doors and windows :

1. Frame. It is an assembly of horizontal and vertical members, forming an enclosure, to which the shutters are fixed.

2. Shutters. These are the openable parts of a door or window. It is an assembly of styles, panels and rails.

3. Head. This is the top or uppermost horizontal part of a frame.

4. Sill. This is the lowermost or bottom horizontal part of a window frame. Sills are normally not provided in door frames.

5. Horn. These are the horizontal projections of the head and sill of a frame to facilitate the fixing of the frame on the wall opening. The length of horns is kept about 10 to 15 cm.

6. Style. Style is the vertical outside member of the shutter of a door or window.

7. Top rail. This is the top most horizontal member of a shutter.

8. Lock rail. This is the middle horizontal member of a door shutter, to which locking arrangement is fixed.

9. Bottom rail. This is the lowermost horizontal member of a shutter.

10. Intermediate or cross-rails. These are additional horizontal rails, fixed between the top and bottom rails of a shutter. A rail fixed between the top rail and lock rail is called *frieze rail.*

11. Panel. This is the area of shutter enclosed between the adjacent rails.

12. Mullion. This is a vertical member of a frame, which is employed to sub-divide a window or a door vertically.

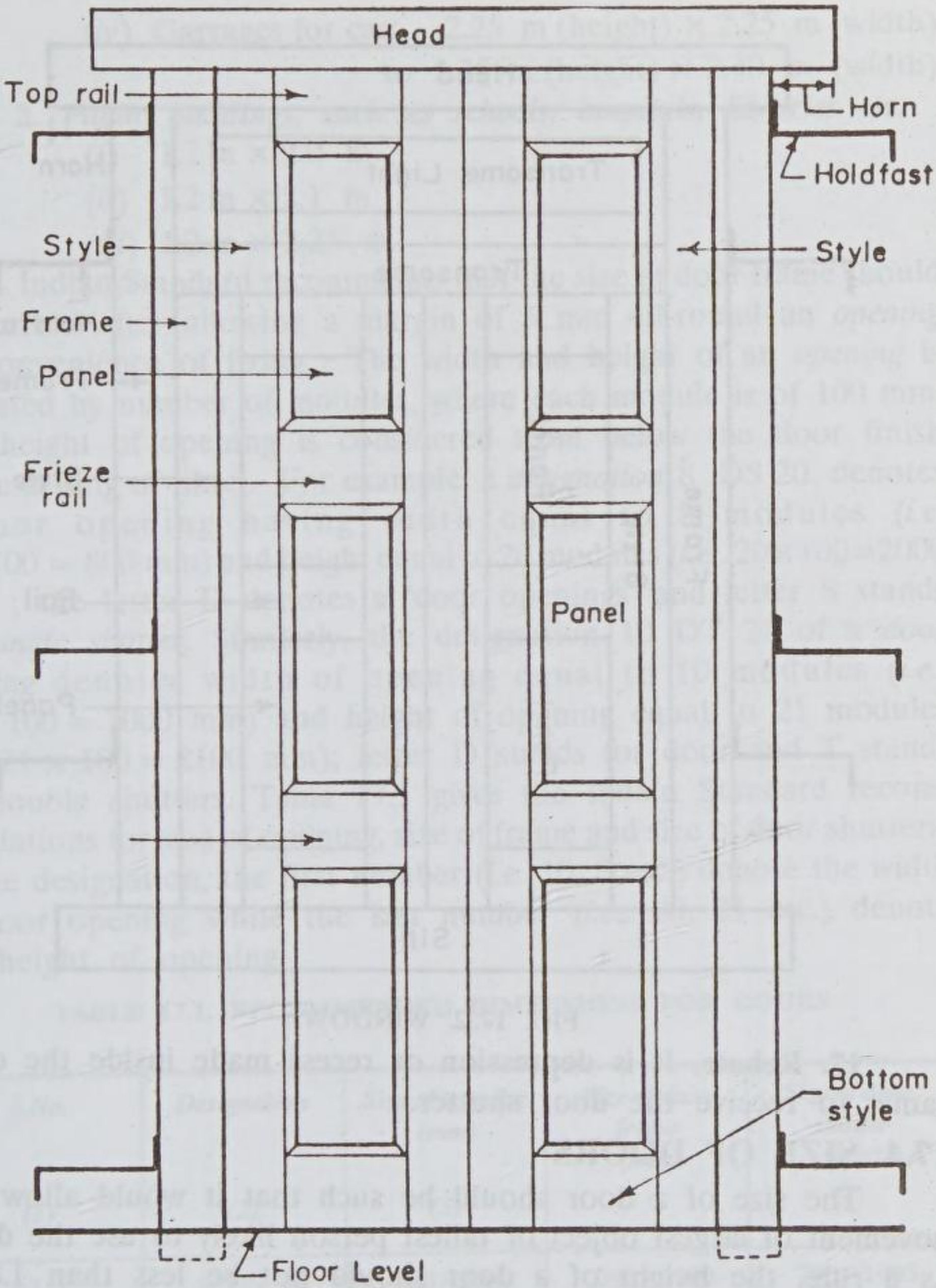


FIG. 17.1. DOOR

13. Transom. This is a horizontal member of a frame, which is employed to sub-divide a window opening horizontally.

14. Hold fasts. These are mild steel flats (section 30 mm × 6 mm), generally bent into Z-shape, to fix or hold the frame to the opening. The horizontal length of hold fast is kept about 20 cm, and is embedded in the masonry.

15. Jamb. This is the vertical wall face of an opening which supports the frame.

16. Reveal. It is the external jamb of a door or window opening at right angles to the wall face.

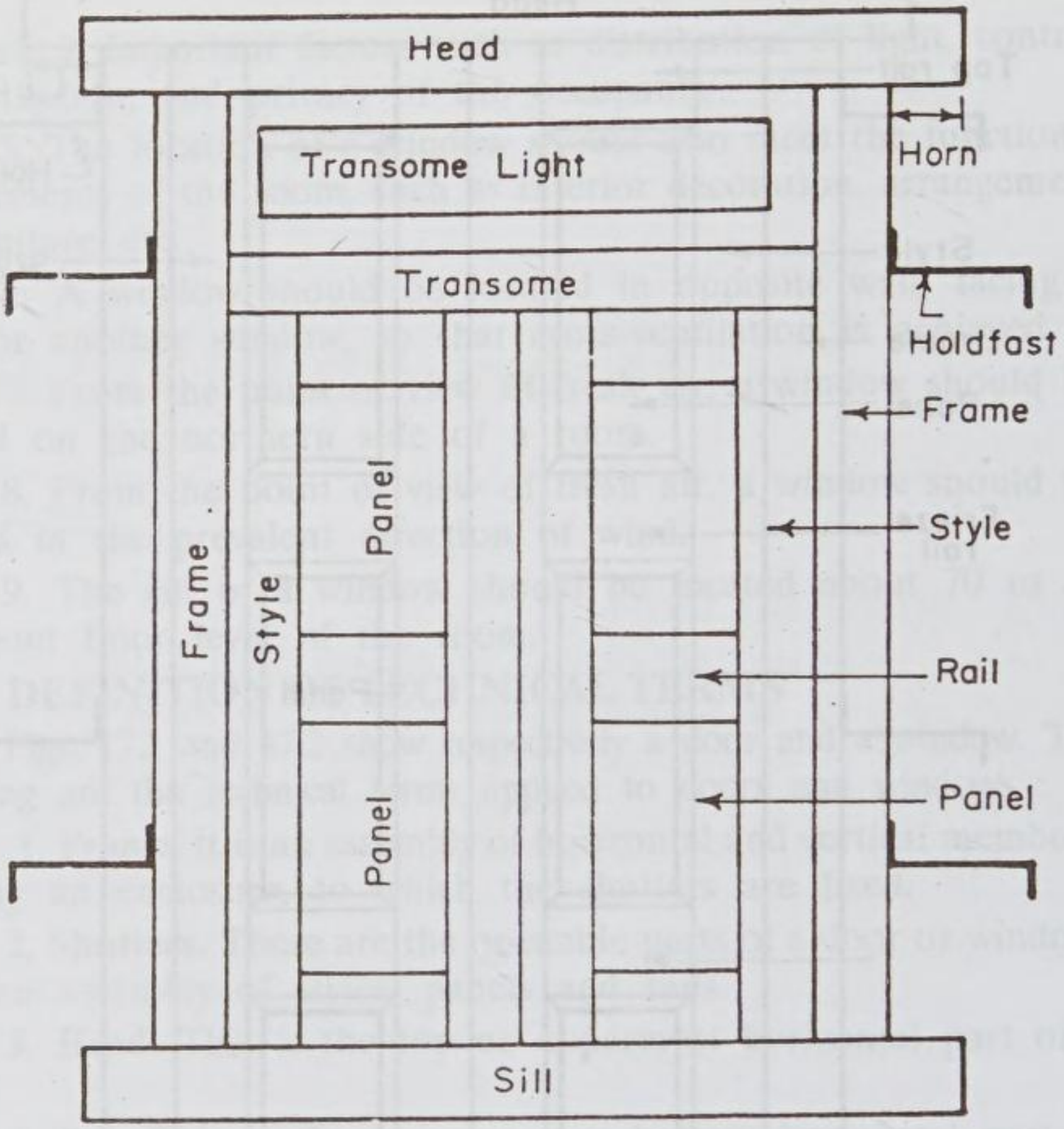


FIG. 17..2. WINDOW.

17. Rebate. It is depression or recess made inside the door frame, to receive the door shutter.

17.4. SIZE OF DOORS

The size of a door should be such that it would allow the movement of largest object or tallest person likely to use the door. As a rule, the height of a door should not be less than 1.8 m to 2 m. The width of the door should be such that two persons can pass through it walking shoulder to shoulder. The common width-height relations, used in India are :

(*i*) Width = 0.4 to 0.6 height.

(*ii*) Height = (width + 1.2) metres.

The following are the generally adopted sizes of doors for various types of buildings :

1. ***Doors of residential buildings***

(*i*) External door ... (1.0 m × 2 m) to (1.1 m × 2 m)

(*ii*) Internal door ... (0.9 m × 2 m) to (1 m × 2 m)

(*iii*) Doors for bathrooms and water closets ... (0.7 m × 2 m) to (0.8 m × 2 m)

(*iv*) Garrages for cars ... 2.25 m (height) × 2.25 m (width) to 2.25 m (height) × 2.40 m (width)

2. *Public buildings, such as schools, hospitals, libraries etc.*

(*i*) 1.2 m × 2.0 m

(*ii*) 1.2 m × 2.1 m

(*iii*) 1.2 m × 2.25 m.

Indian Standard recommends that the size of door frame should be derived after allowing a margin of 5 mm all-round an *opening* for convenience of fixing. The width and height of an *opening* is indicated by number of modules, where each module is of 100 mm. The height of opening is considered from below the floor finish to the ceiling of lintel. For example, a *designation* 8 DS 20 denotes a door opening having width equal to 8 modules (*i.e.* 8 × 100 = 800 mm) and height equal to 20 modules (*i.e.*. 20×100=2000 mm) ; the letter D denotes a 'door openings' and letter S stands for *single shutter*. Similarly, the designation 10 DT 21 of a *door opening* denotes width of opening equal to 10 modules (*i.e.*. 10 × 100 = 1000 mm) and height of opening equal to 21 modules (*i.e.* 21 × 100 = 2100 mm); letter D stands for door and T stands for double shutters. Table 17.1 gives the Indian Standard recommendations for size of opening, size of frame and size of door shutters. In the designation, the first number (*i.e.* 10, 8 etc.) denote the width of door opening while the last number (*i.e.*, 20, 21 etc.) denote the height of opening.

TABLE 17.1. RECOMMENDED DIMENSIONS FOR DOORS

S.No. (1)	*Designation* (2)	*Size of opening (mm)* (3)	*Size of door frame (mm)* (4)	*Size of door shutter (mm)* (5)
1.	8 DS 20	800×2000	790×1990	700×1905
2.	8 DS 21	800×2100	790×2090	700×2005
3.	9 DS 20	900×2000	890×1990	800×1905
4.	9 DS 21	900×2100	890×2090	800×2005
5.	10 DT 20	1000×2000	990×1990	900×1905
6.	10 DT 21	1000×2100	990×2090	900×2005
7.	12 DT 20	1200×2000	1190×1990	*1000×1905
8.	12 DT 21	1200×2100	1190×2090	*1000×2005

*500 mm each shutter and 20 mm overlap when closed.

The thickness of shutters shall be 20, 25 or 30 mm depending upon size.

17.5. DOOR FRAMES

A door frame is an assembly of horizontal and vertical members forming an enclosure, to which door shutters are fixed. The vertical members (one to each side) are known as *jambs* or *posts*, while the horizontal top member connecting the posts is called the *head* which has horns to both the sides. The size of the frame is determined by allowing a clearance of 5 mm to both the sides and the top of the opening. The cross-sectional area of the *posts* and the *head* is generally kept the same. Fig. 17.3. shows the general view of a door frame, having a *rebate* cut all-round it to receive door shutter.

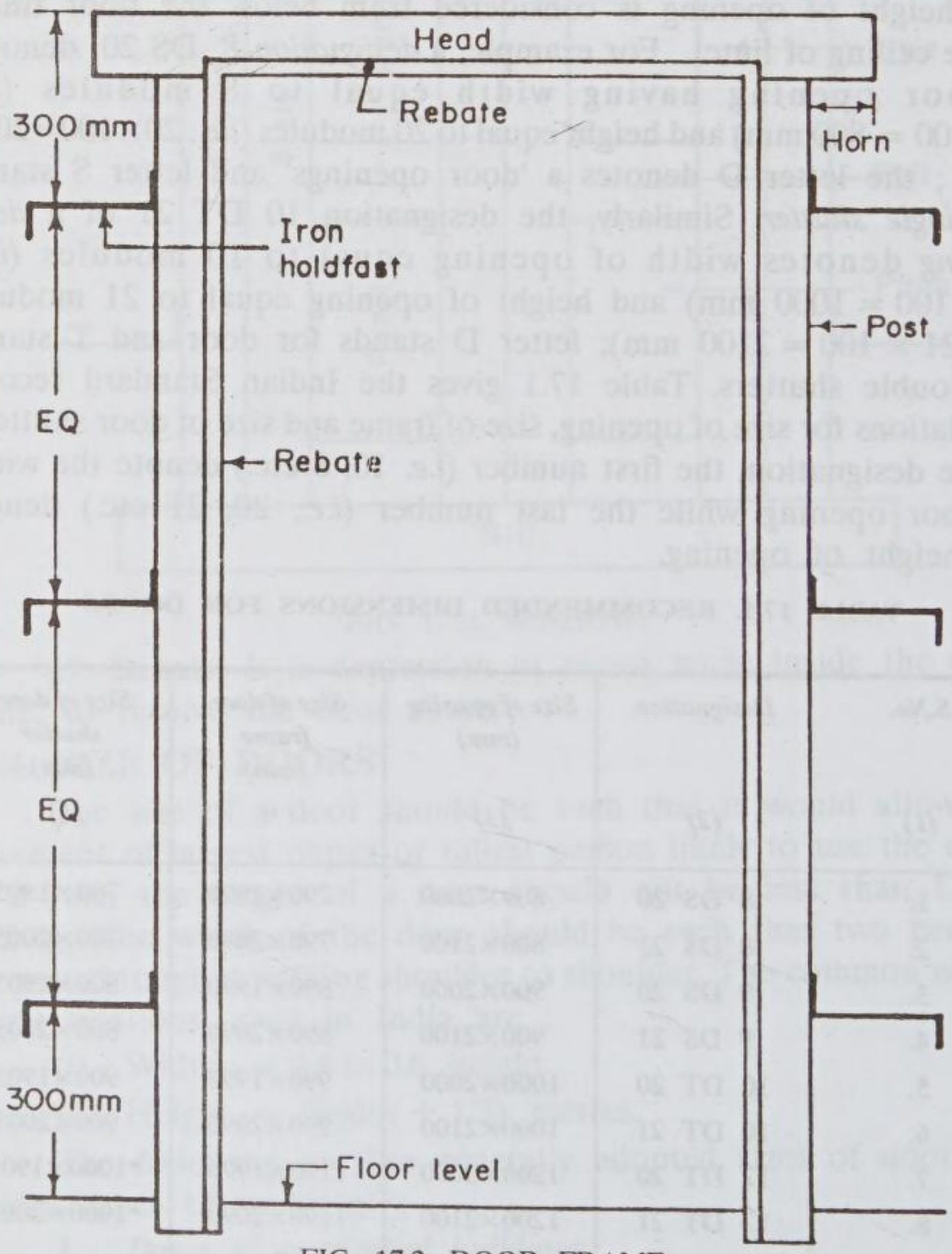


FIG. 17.3. DOOR FRAME.

Door frames are made of following materials.

(*i*) Timber.

(*ii*) Steel sections.

(*iii*) Aluminium sections
(*iv*) Concrete
and (*v*) Stone.

Out of these, *timber frames* are more commonly used. However, in factories, workshops, etc., *steel frames* are widely used. *Aluminium frames* are costlier and are used only for residential buildings where more funds are available. With the increasing cost of timber, and with the increasing menace of *termites* (white ants), *concrete frames* are now becoming popular in urban areas. Concrete frames are found to cost less than half the cost of a teakwood frame; it is also cheaper than iron frame. *Stone frames* are used mostly in villages, specially where good quality stone is available, in required sizes. The jambs or posts, head and sill of the stone door frame are finely dressed, and are jointed by forming proper holes and corresponding projections.

TIMBER DOOR FRAME

Generally, timber door frames are preferred because they look much better than other materials, and they can be polished, if desired. The thickness of timber frame varies from 60 to 75 mm, depending upon the size of the door opening and the type of timber used. The same thickness is used for jambs as well as head. The width

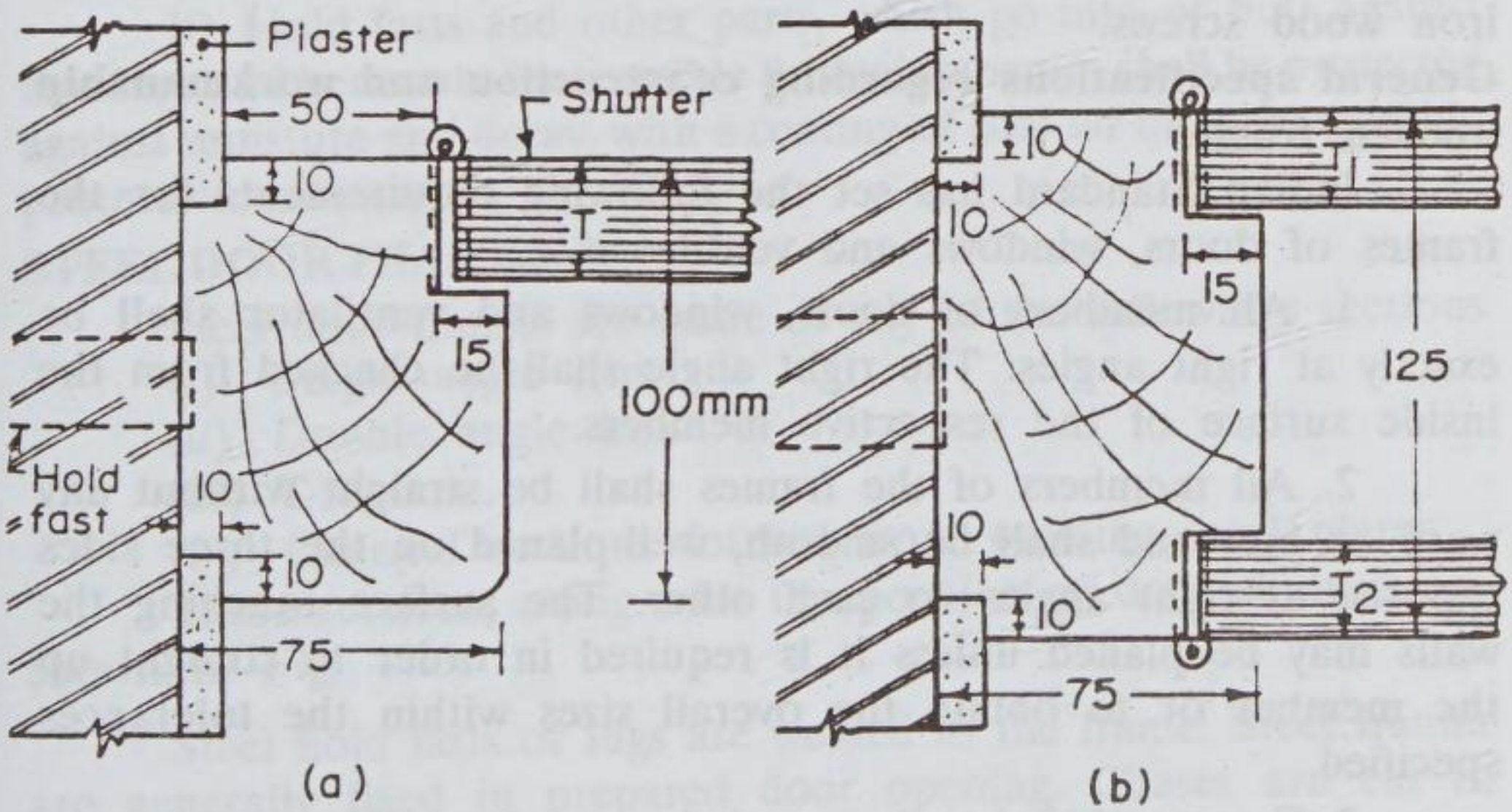


FIG. 17.4. TIMBER DOOR FRAMES.

of the door frame is taken as 100 mm if the door has shutter to one side only (which is the general case), and 125 to 140 mm if shutters are provided to both the sides of the frame (such as panelled shutter to one side and fly proof wire-mesh shutter to the other side). Fig. 17.4 shows, in the plan, the cross-section of jambs of timber door frames having shutter to one side or both the sides.

The frame is grooved on both the inner edges to receive plaster,

the depth of rebating being equal to 10 mm. In ordinary works, this grooving is avoided. Similarly, the frame is rebated at its outer edge to receive the shutter. The width of rebating varies from 12.5 to 15 mm while the depth is kept equal to the thickness T of the shutter. In case the door has shutters to both the sides, rebating is also done on both the edges of each jamb and the head.

The joint between vertical post and the head of the frame may be of the following types :

(*i*) Closed mortised and tenoned joint.

(*ii*) Pin and tenoned joint.

(*iii*) Dovetailed joint.

Indian Standard recommends a dovetail joint, with dovetail in the post and recess in the head of the frame.

Method of fixing

Before fixing the door frame, all the portions of the frame which are likely to come in contact with masonry are painted with coal tar mixed with 'aldrex' (anti-termite solution), or with any approved wood primer. The hold fasts, attached to the frame, are well-embedded in masonry, with concrete around the hold fasts. In case the frame is to be fixed later, wooden pegs or plugs are embedded in the masonry, with their ends flushing with the face of the opening. The door frame is later screwed to these pegs or plugs through galvanised iron wood screws.

General specifications regarding construction and workmanship wooden frames

Indian Standard has set the following requirements for the frames of doors, windows and ventilators :

1. All members of doors, windows and ventilator shall be exactly at right angles. The right angle shall be checked from the inside surface of the respective members.

2. All members of the frames shall be straight without any warp or bow and shall be smooth, well-planed on the three sides exposed at right angles to each other. The surface touching the walls may be planed unless it is required in order to straight up the member or to obtain the overall sizes within the tolerances specified.

3. Frames of timber doors, windows and ventilators shall have dovetailed joints.

4. The jamb post shall be though-tenoned into the mortices of the transom to the full width of the transom and the thickness of the tenon shall be not less than 15 mm. The tenons shall be closely fitting into the mortices and pinned with corrosion resisting star-shaped metal pins not less than 8 mm in diameter, or with wood dowels not less than 10 mm diameter. The depth of rebate

in frames for housing the shutters shall be 15 mm.

5. Members of frames of doors, windows and ventilators shall be of the same species of timber except in case of soft wood frames where the bottom sill of the window and ventilator frames shall be of hard wood.

6. The contact surfaces of tenons and mortices shall be treated before putting together with proper adhesive or animal glue or polyvinyl acetate dispersion based adhesive.

7. A minimum of three hold fasts shall be fixed on each side of door and window frames, one at the centre point and the other two at 300 mm from top and the bottom of the frames. In case of window and ventilator frames whose height is less than 1 m, two hold fasts on each side shall be fixed at quarter points of the frames. Unless otherwise specified, these will be of Z form, 50 mm ×6 mm and 230 mm long, fixed with specified screws for door and window frames.

8. The frames shall be well planned on the three sides exposed at right angles to each other and finished smooth.

9. The frames shall be clamped together so as to be square and flat at the time of delivery. Each assembled door frame shall be fitted with temporary stretchers.

10. Hold fasts and other parts, which go into or butt against masonry and hence are inaccessible for maintenance, shall be protected against moisture and decay, with a coating of coal tar or other suitable protective material.

STEEL DOOR FRAMES

Steel door frames are made of any of the followings sections

(*i*) Single angle iron
(*ii*) Double angle iron
(*iii*) T-section
(*iv*) Channel sections formed from pressing steel plates.

These sections, along with the position of door shutters are shown in Fig. 17.5.

Steel hold fasts or lugs are welded to the frame. Steel frames are generally fixed in prepared door opening. Chases are cut in brick masonry for accommodating hold fasts or lugs which are then grouted with cement mortar. The vertical jambs and the head of the frame are welded together. The hinges of the shutters are also welded to the frame. In the case of stone masonry or R.C.C. where it is difficult to cut chases, wooden plugs are embedded at appropriate places in the jamb during the construction of wall. The steel frame is fixed with plugs with the help of galvanised iron wood screws of big size.

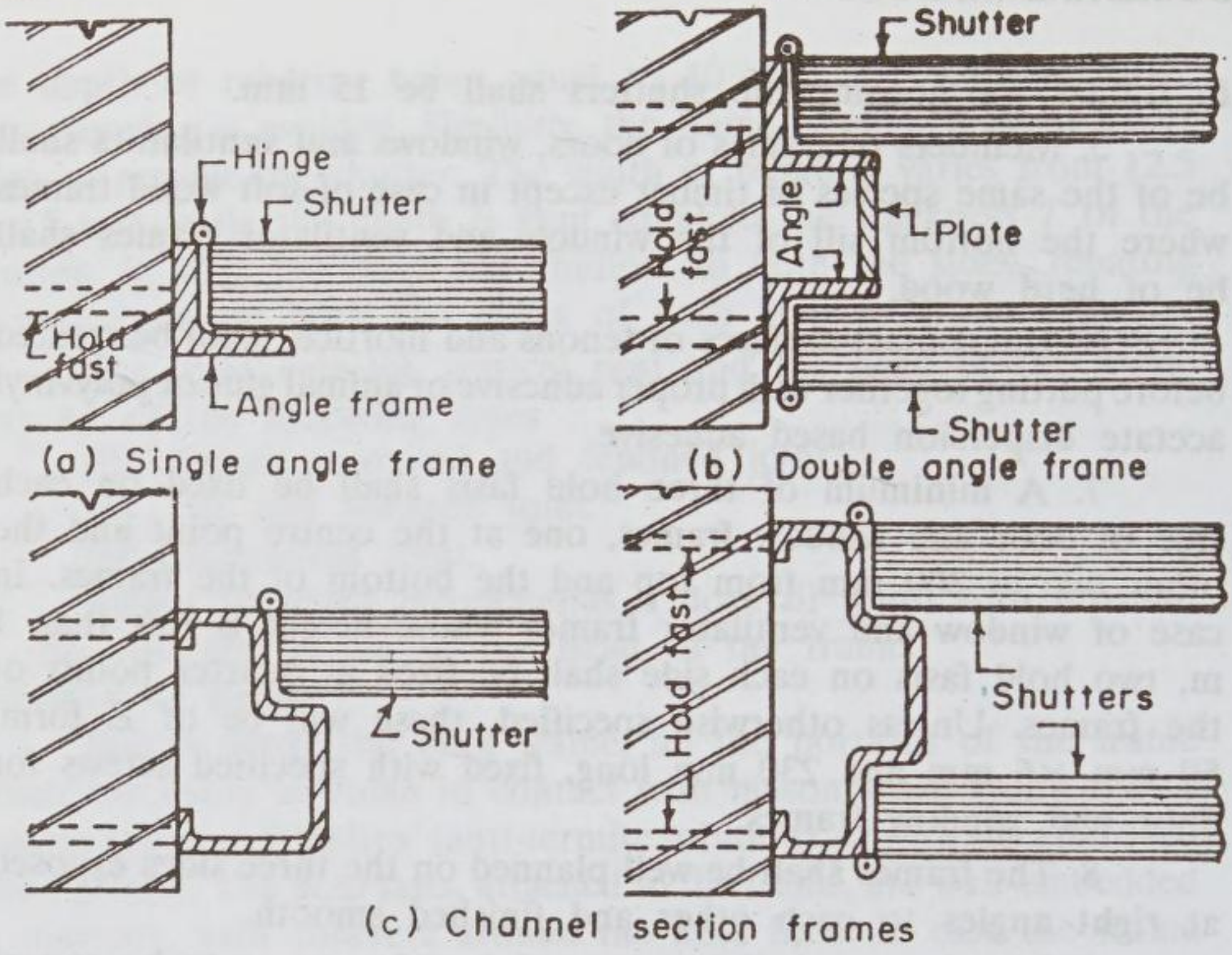


FIG. 17.5. STEEL DOOR FRAMES.

17.6. TYPES OF DOORS

Doors commonly used in building are classified into the following types, depending upon (*i*) type of materials used (*ii*) arrangement of different components of the door, (*iii*) method of construction, and (*iv*) nature of working operations:

1. Battened and ledged doors 2. Battened, ledged and braced doors 3. Battened, ledged and framed doors 4. Battened, ledged, braced and framed doors	*Classification on the basis of arrangement of components*
5. Framed and panelled doors 6. Glazed or sash doors 7. Flush doors 8. Louvered doors 9. Wire-gauged doors	*Classification on the basis of method or manner of construction*
10. Revolving doors 11. Sliding doors 12. Swing doors 13. Collapsible steel doors 14. Rolling steel shutter doors	*Classification on the basis of working operations*
15. Mild steel sheet doors 16. Corrugated steel sheet doors 17. Hollow metal doors 18. Metal covered plywood doors.	*Metal doors.*

1. BATTENED AND LEDGED DOORS

This is the simplest type of door, specially suitable for narrow openings. The door, shown in Fig. 17.6 is formed of vertical bonds, known as *battens*, which are usually tongued and grooved, and are fixed together by horizontal supports known as *ledges*. Battens are

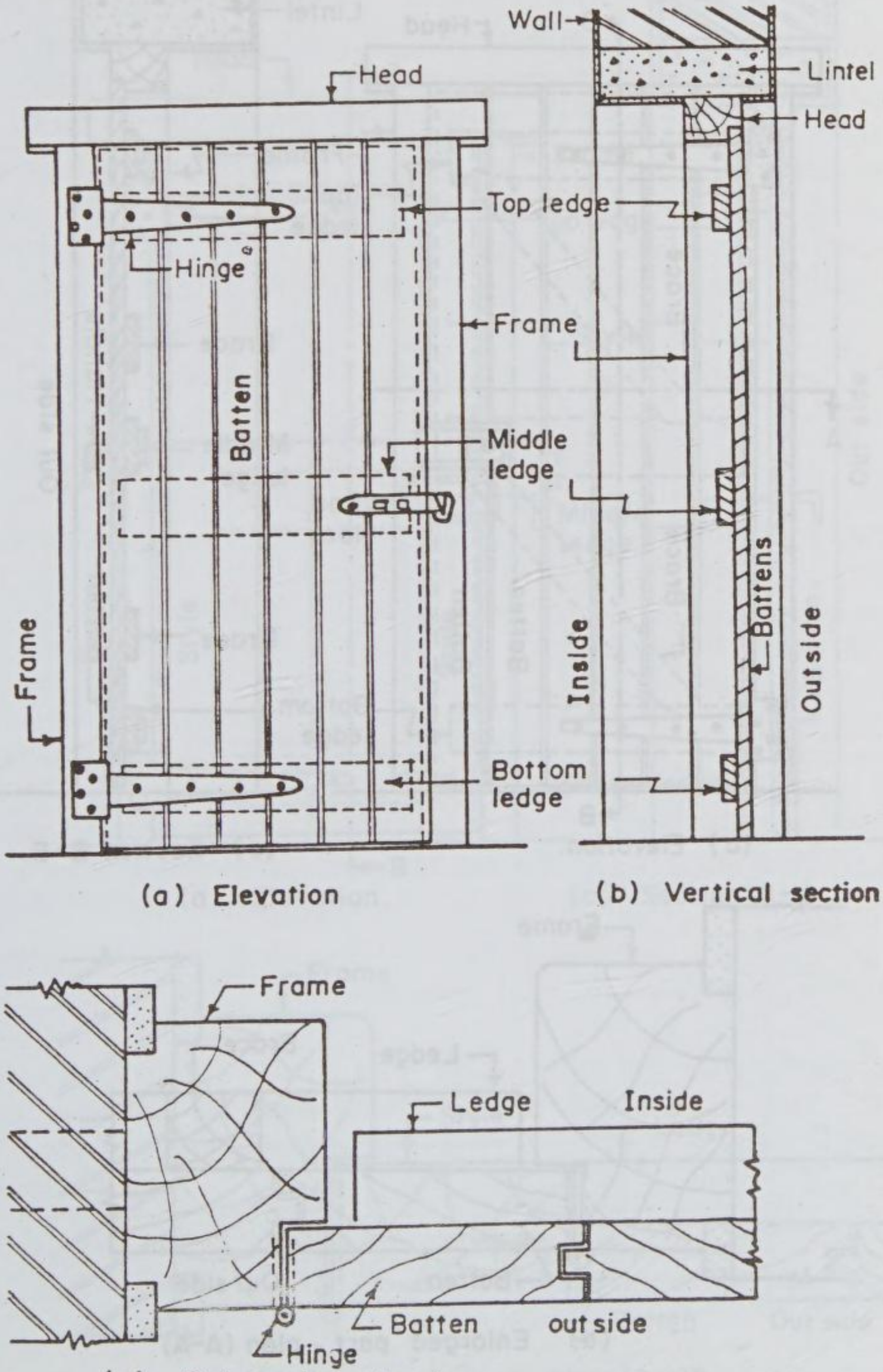


FIG. 17.6. BATTENED AND LEDGED DOOR.

100 to 150 mm wide and 20 to 30 mm thick. Ledges are 100 to 200 mm wide and 25 to 30 mm thick. Three ledges are generally provided—top, middle and bottom. The door is hung to the frame by means of T-hinges of iron.

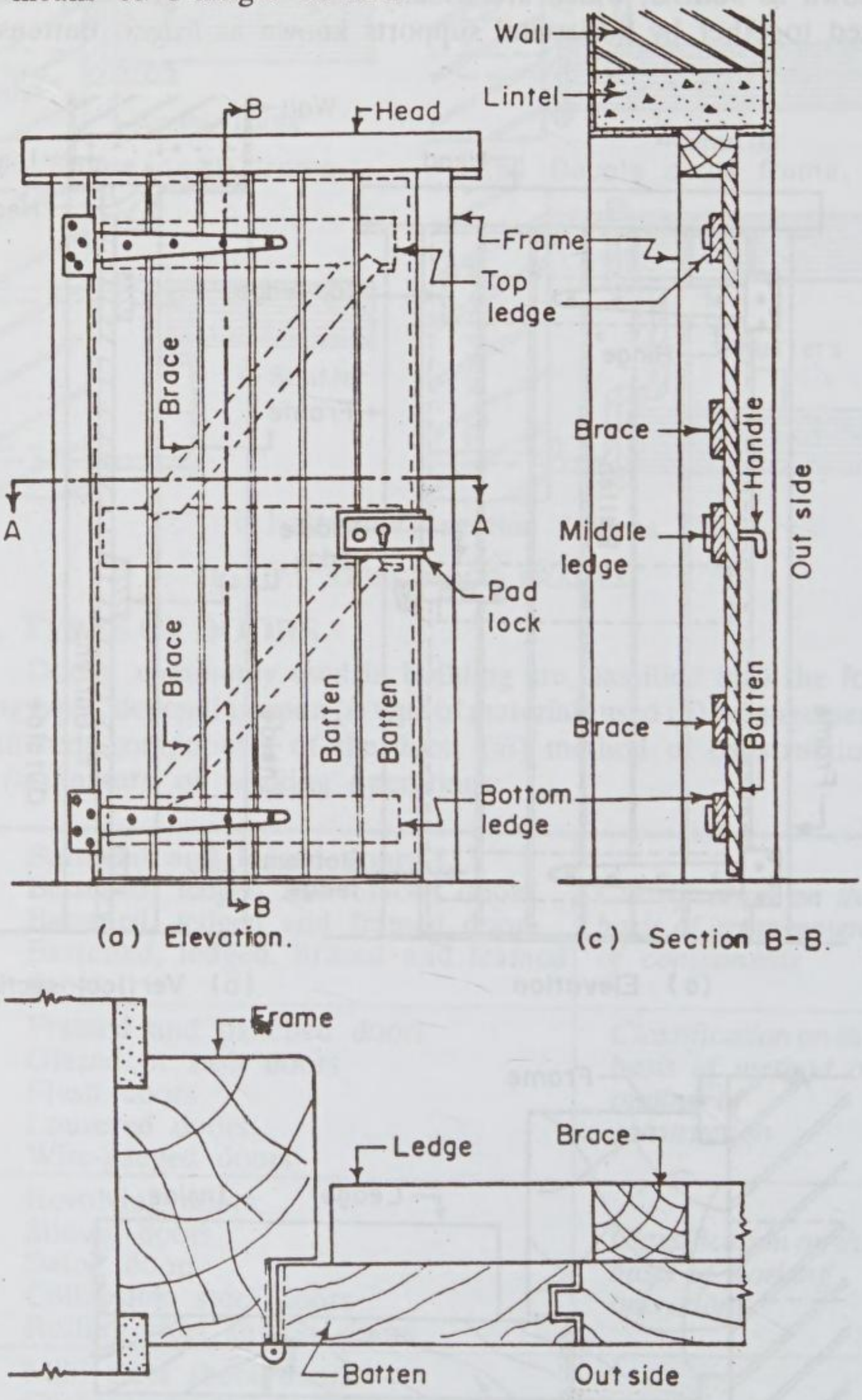


FIG. 17.7. BATTENED, LEDGED AND BRACED DOOR.

2. BATTENED, LEDGED, AND BRACED DOORS

These doors are improved versions of battened and ledged doors, in which additional inclined (or diagonal) members, called *races* are provided, as shown in Fig. 17.7, to give more rigidity. Hence these doors can be used for wider openings. The braces, 100

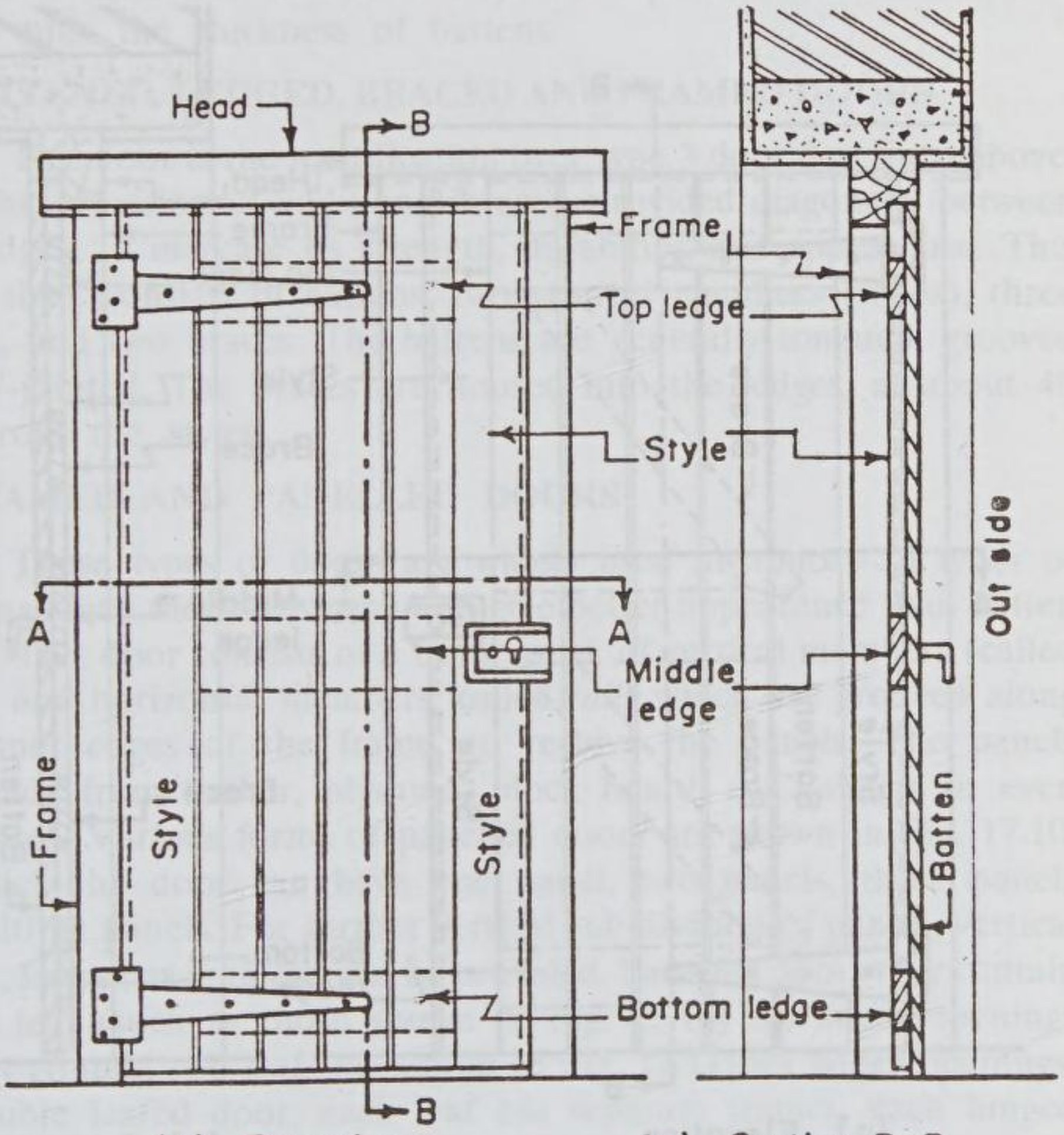


FIG. 17.8. BATTENED, LEDGED AND FRAMED DOOR.

to 150 mm wide have the same thickness as the ledges, and are simply housed in the ledges. It is essential that the braces slope upwards from the handing side since they have to work as struts, to take compression.

3. BATTENED, LEDGED AND FRAMED DOORS

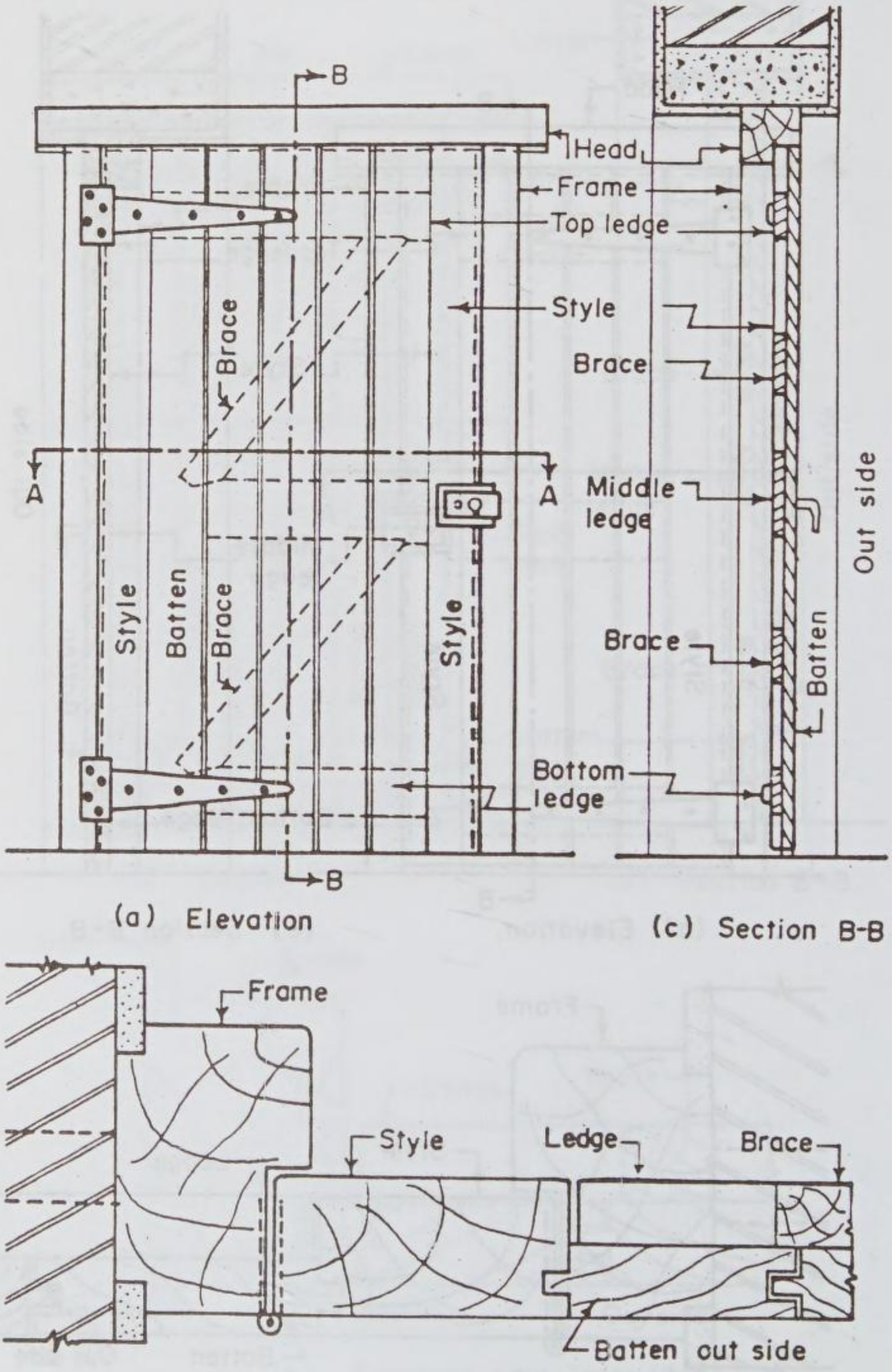


FIG. 17.9. BATTENED, LEDGED, BRACED AND FRAMED DOOR.

This door is also an improved form of simple battened and ledged door, in which frame work for the shutter is provided in the form of two verticals, known as *styles*. Styles are generally 100 mm wide and 40 mm thick. Three ledges are provided as usual. The total thickness of *style* is adjusted equal to the thickness of ledges plus the thickness of battens.

4. BATTENED, LEDGED, BRACED AND FRAMED DOORS

This door is the modification over type 3 door described above, with the provision of additional braces, provided diagonally between the ledges, to increase its strength, durability, and appearance. This door, thus, consists of battens, two vertical members (styles), three ledges, and two braces. The battens are generally tongued, grooved and V-jointed. The braces are housed into the ledges, at about 40 mm from the styles.

5. FRAMED AND PANELLED DOORS

These types of doors are widely used in almost all types of building since they are strong and give better appearance than batten doors. This door consists of a frame work of vertical members (called *styles*) and horizontal members, called *rails* which are grooved along the inner edges of the frame, to receive the panels. The panels are made from timber, plywood, block board, A.C. sheets or even of glasses. Various forms of panelled doors are shown in Fig. 17.10, in which the door can have one panel, two panels, three panels or multiple panels. For further vertical sub-division of panels, vertical pieces, known as *mullions* can be provided. Panelled doors may contain single leaf (such as those shown in Fig. 17.10) for small openings or may contain two leafs (as shown in Fig. 17.11) for wider openings. In double leafed door, each leaf has separate frames, each hinged to the corresponding jamb-post of the door.

Salient features of framed and panelled doors

1. The styles are continuous from top to bottom, *i.e.* they are in single piece.
2. Various rails (*i.e.* top rail, bottom rail and intermediate rails) are jointed to the styles at both the ends.
3. The styles and the rails are jointed by tenon and mortised joints.
4. Mullions or mutines, if provided, are jointed to the adjacent rails between which it is fixed.
5. The bottom and lock rails are made wider than top and frieze rails.
6. The entire frame is grooved on all the inside faces to receive the panels.

7. Additional timber *beading* is provided either on one or on both the sides to improve the elevation of the door.

8. The lock rail elevation is so adjusted that its centre line is at a height of about 800 mm from the bottom of the shutter.

9. The minimum width of style is kept as 100 mm. The minimum width of bottom rail and lock rail is kept as 150 mm.

10. If panels are made of timber, its minimum width should be 150 mm, and minimum thickness should be 15 mm. However, the maximum area of single panel of timber should not be more than 0.5 m^2. These restrictions do not apply to panels of plywood, particle board or hard board.

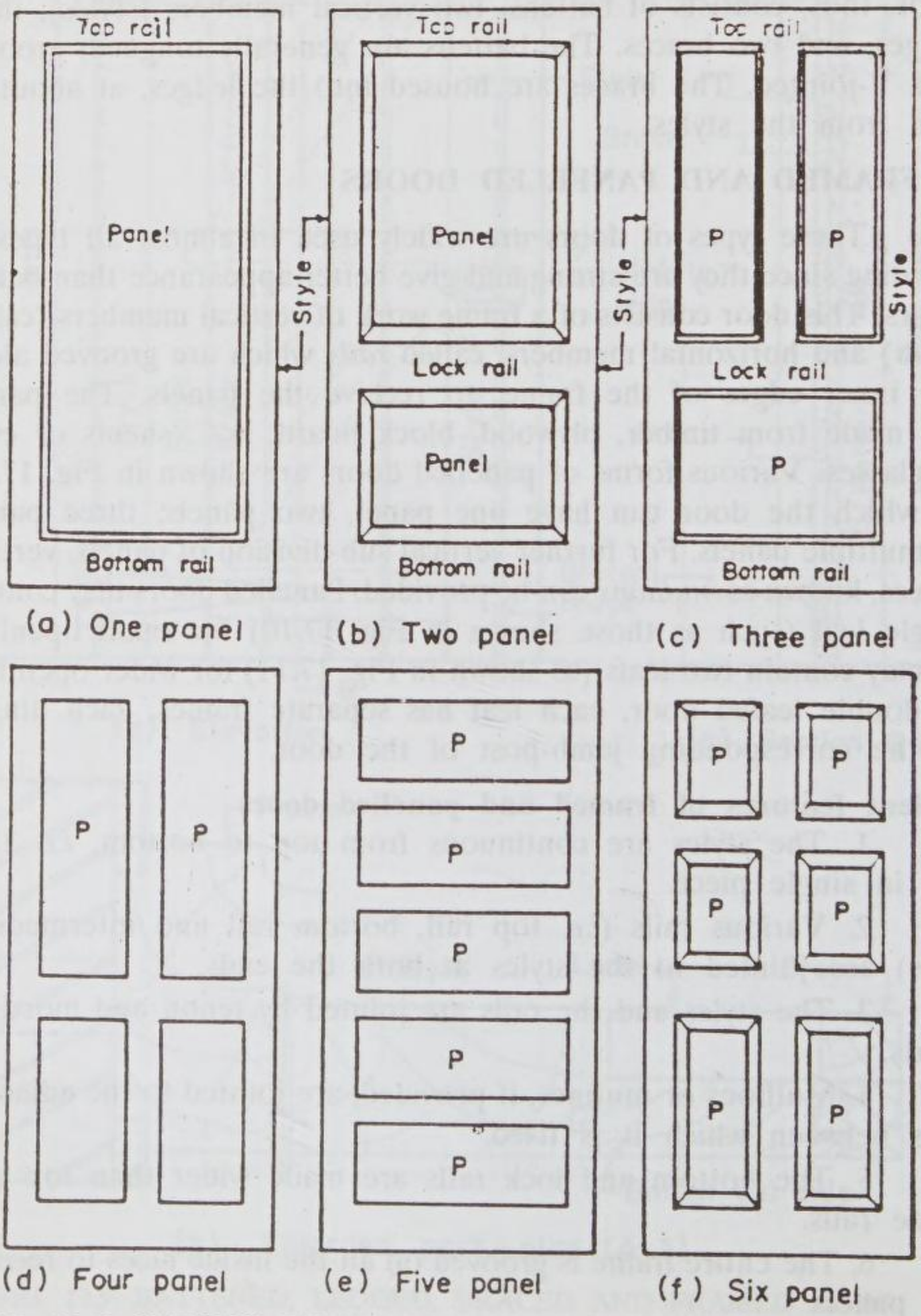


FIG. 17.10. VARIOUS FORMS OF SINGLE-LEAF PANELLED DOORS.

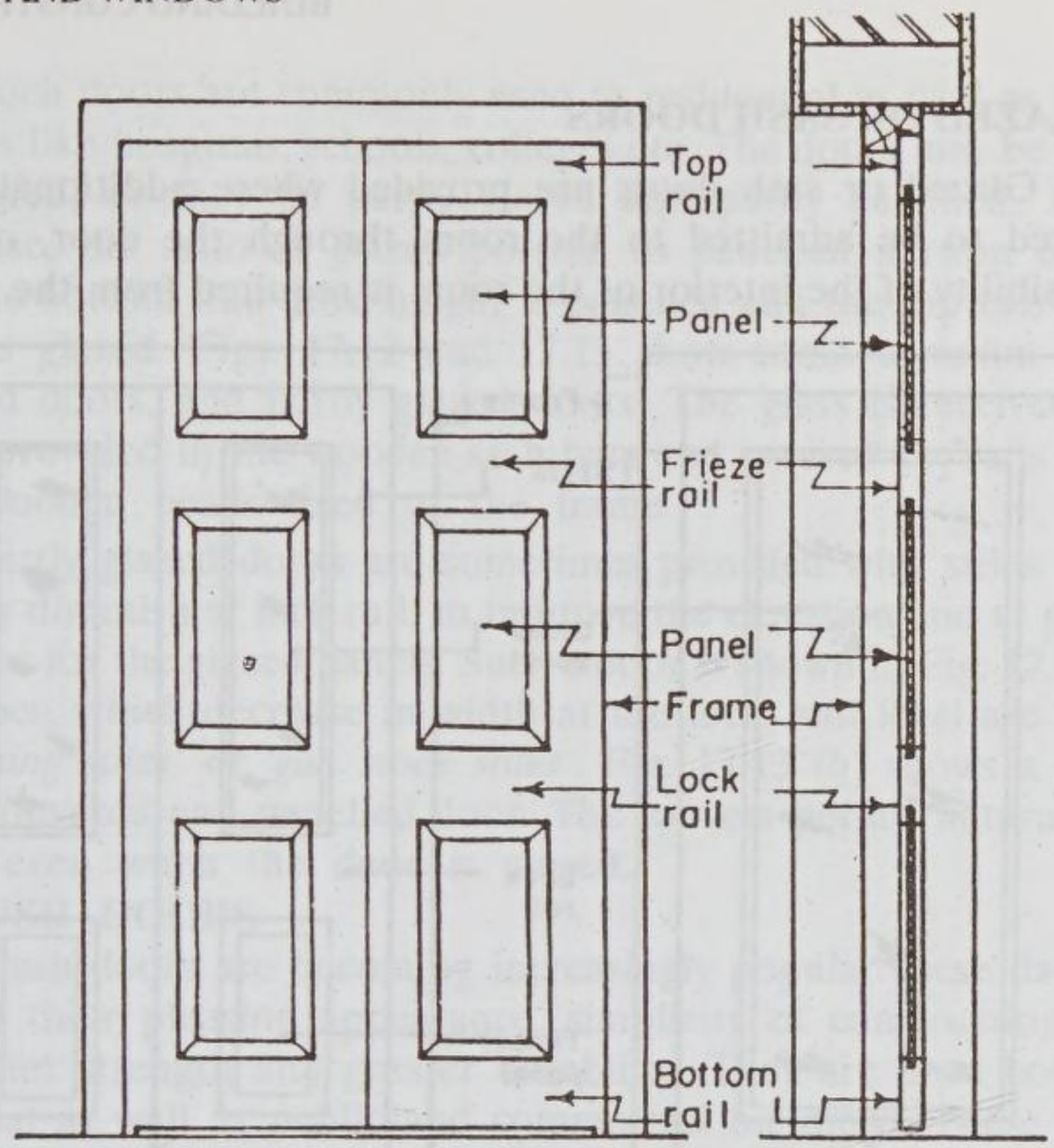
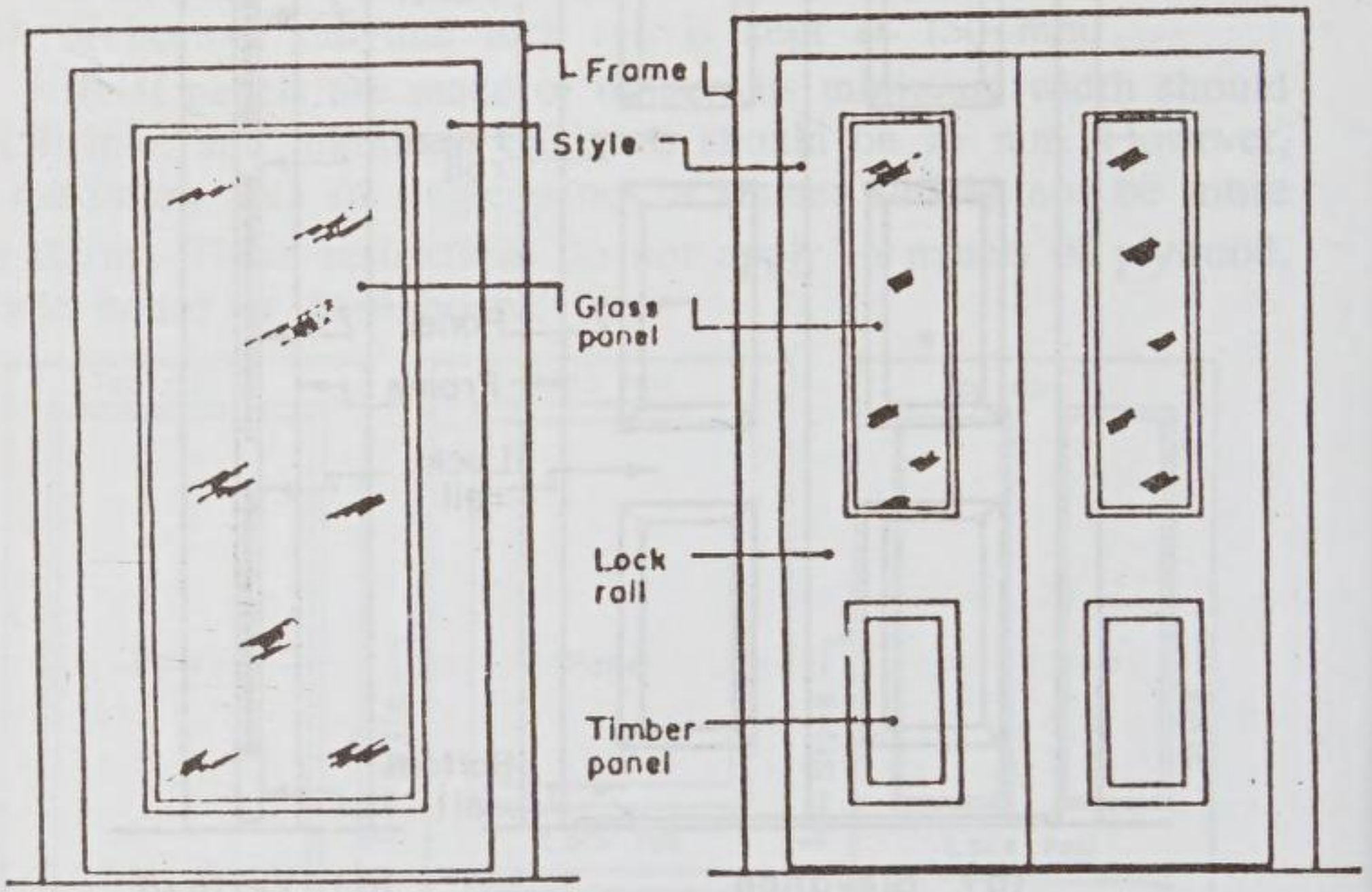


FIG. 17.11. DETAILS OF A DOUBLE--LEAF SIX PANELLED DOOR.

6. GLAZED OR SASH DOORS

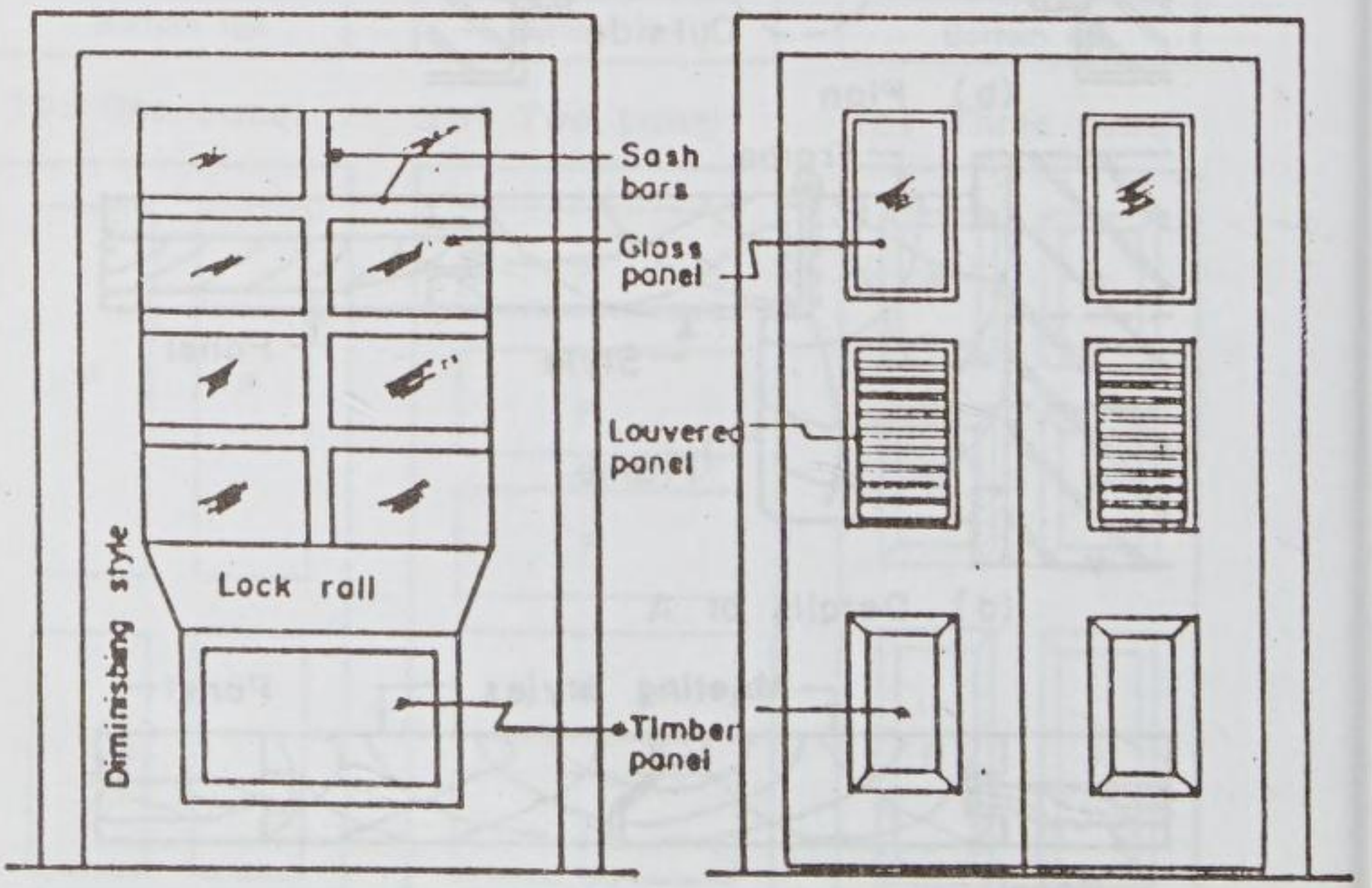
Glazed or sash doors are provided where additional light is required to be admitted to the room through the door, or where the visibility of the interior of the room is required from the adjacent



(a) Fully glazed single-leaf door

(b) Partly glazed, partly panelled double leaf door

FIG. 17.12. FULLY AND PARTLY GLAZED DOORS.



(a) Party glazed door with diminishing style

(b) Partly glazed, louvered and panelled door

FIG 17.13. FULLY AND PARTLY GLAZED DOORS.

room. Such doors are commonly used in residential as well as public buildings like hospitals, schools, colleges etc. The doors may be either fully glazed, or they be partly glazed and partly panelled. In the latter case, the ratio of glazed portion to panelled portion is kept 2 : 1 ; the bottom one-third height is panelled and the top two-thirds height is glazed. Figs. 17.12 and 17.13 show some common forms of glazed doors, and partly glazed doors. The glass is received into rebates provided in the wooden sash bars and secured by 'rails putty' or by wooden beads fixed to the frame.

Partly glazed doors are sometimes provided with stiles which gradually diminish at lock rail, to improve the elevation and to permit more area for the glazed panels. Such a door is shown in Fig. 17.13(*a*). Such types, which decrease in width at the lock- rail level are called *'diminishing stiles'* or *'gun stock stiles'*. Fig. 17.13 (*b*) shows a partly glazed, louvered and panelled door. The louvers permit natural ventilation even when the door is closed.

7. FLUSH DOORS

Flush doors are becoming increasingly popular these days because of their pleasing appearance, simplicity of construction, less cost, better strength and greater durability. They are used both for residential as well as public and commercial buildings. These doors consist of solid or semi-solid skeleton or core covered on both sides with plywood, face veneers etc., presenting flush and jointless surface which can be neatly polished.

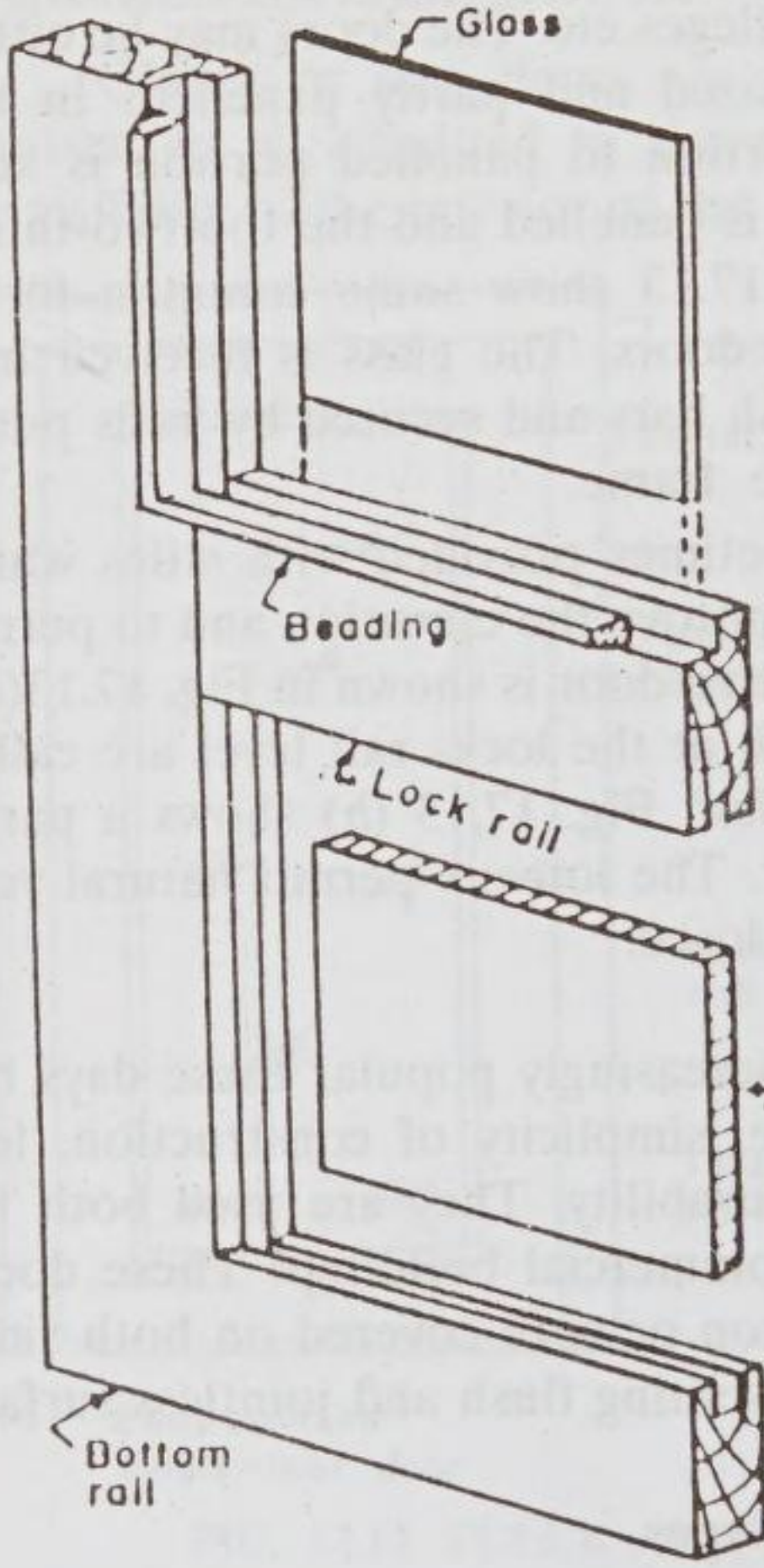
Flushed doors are of two types :

(*i*) Solid core flush door or laminated core flush door.

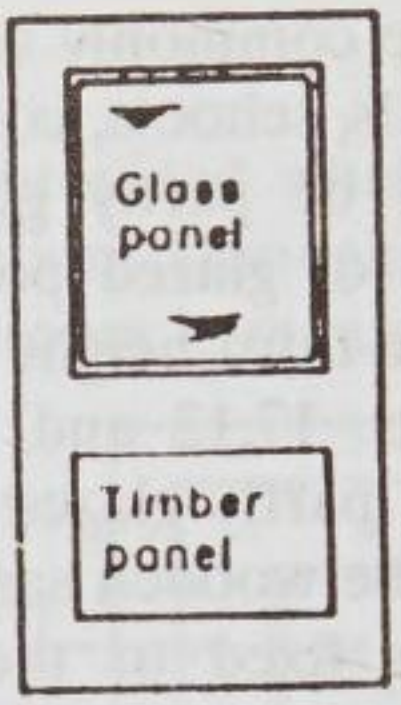
(*ii*) Hollow and cellular core flush door.

(*i*) Solid core flush door or laminated core flush door (Fig. 17.15)

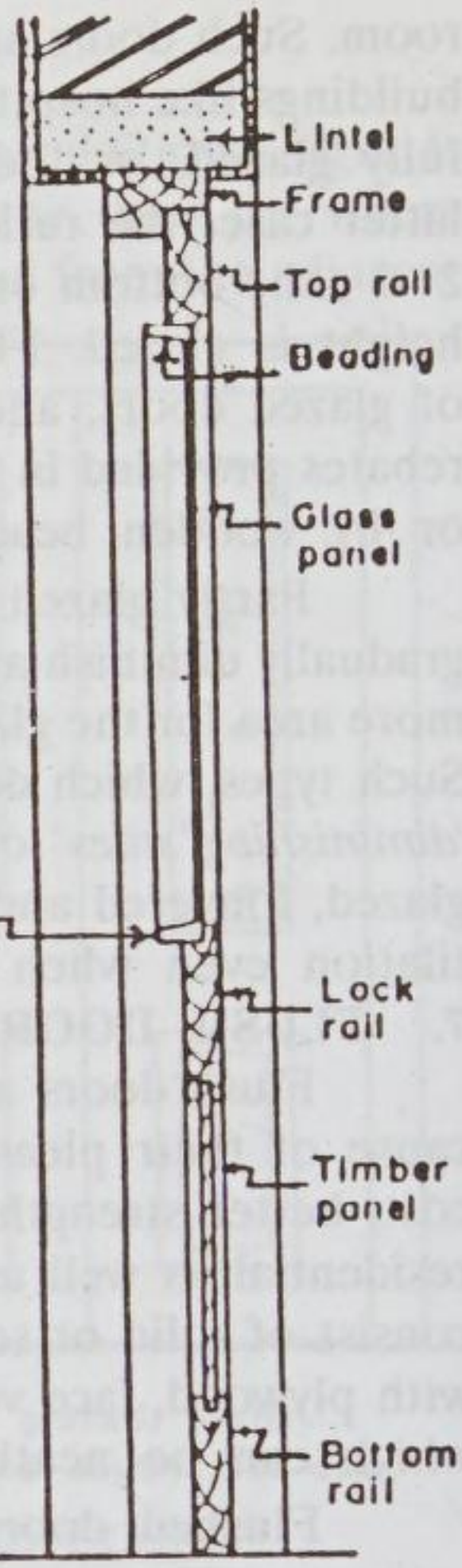
Such a door consists of the wooden frame consisting of styles, and top and bottom rails is used for holding the core. The core consists either of core-strips of timber glued together under great pressure and faced on each side by plywood sheets, or of block board, particle board or a combination of particle board and block board, faced with plywood sheets. In the laminated core, the wooden strips are of maximum width of 25 mm glued together, and the length of each strip is equal to the length of the laminated core. In each type of core, plywood sheets are glued under pressure to the assembly of core housed in the frame on both faces. Alternatively, separate cross-bands and face veneers can be glued on both the faces, with the grains of crossband at right angles to the core and grain of veneer at right angles to that of the cross-band. The core is housed in the outer frame having stiles, top and bottom rails each of not less than 75 mm width. Such doors are quite strong, but are heavy and require more material.



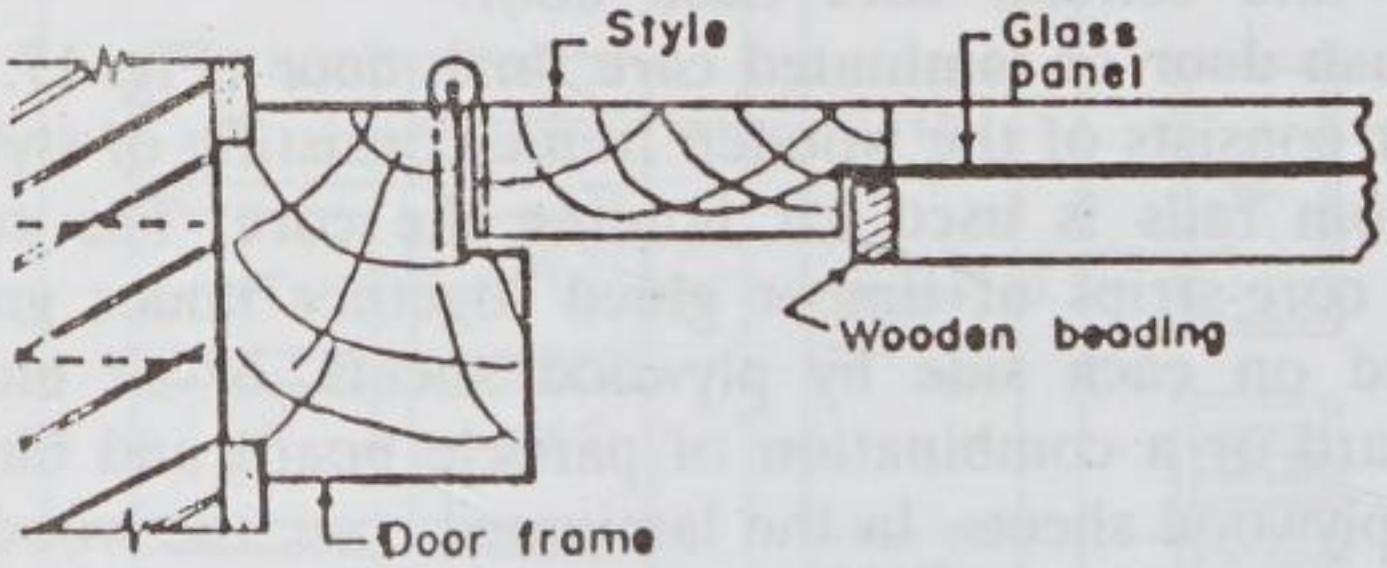
(b) View



(a) Elevation



(d) Vertical section



(e) Enlarged part - plan

FIG. 17.14. DETAILS OF A PARTLY GLAZED SINGLE-LEAF DOOR.

(*ii*) Hollow core and cellular core flush door (Fig. 17.16)

A *hollow core flush door* consists of frame made up of styles, top rail, bottom rail and minimum two intermediate rails, each of a minimum of 75 mm width. The inner space of the frame is provided with equally spaced battens each of minimum 25 mm width, such that the area of voids is limited to 500 sq. cm.

A *cellular core flush door* consists of a frame of styles, top

rail and bottom rail, each of a minimum of 75 mm width, with the void space, filled with equidistant battens of wood or plywood, each of a minimum of 25 mm in width. The battens are so arranged that the void space between adjacent vertical and horizontal battens does not exceed 25 cm^2 in area, and that the total area of voids does not exceed 40% of the area of the shutter.

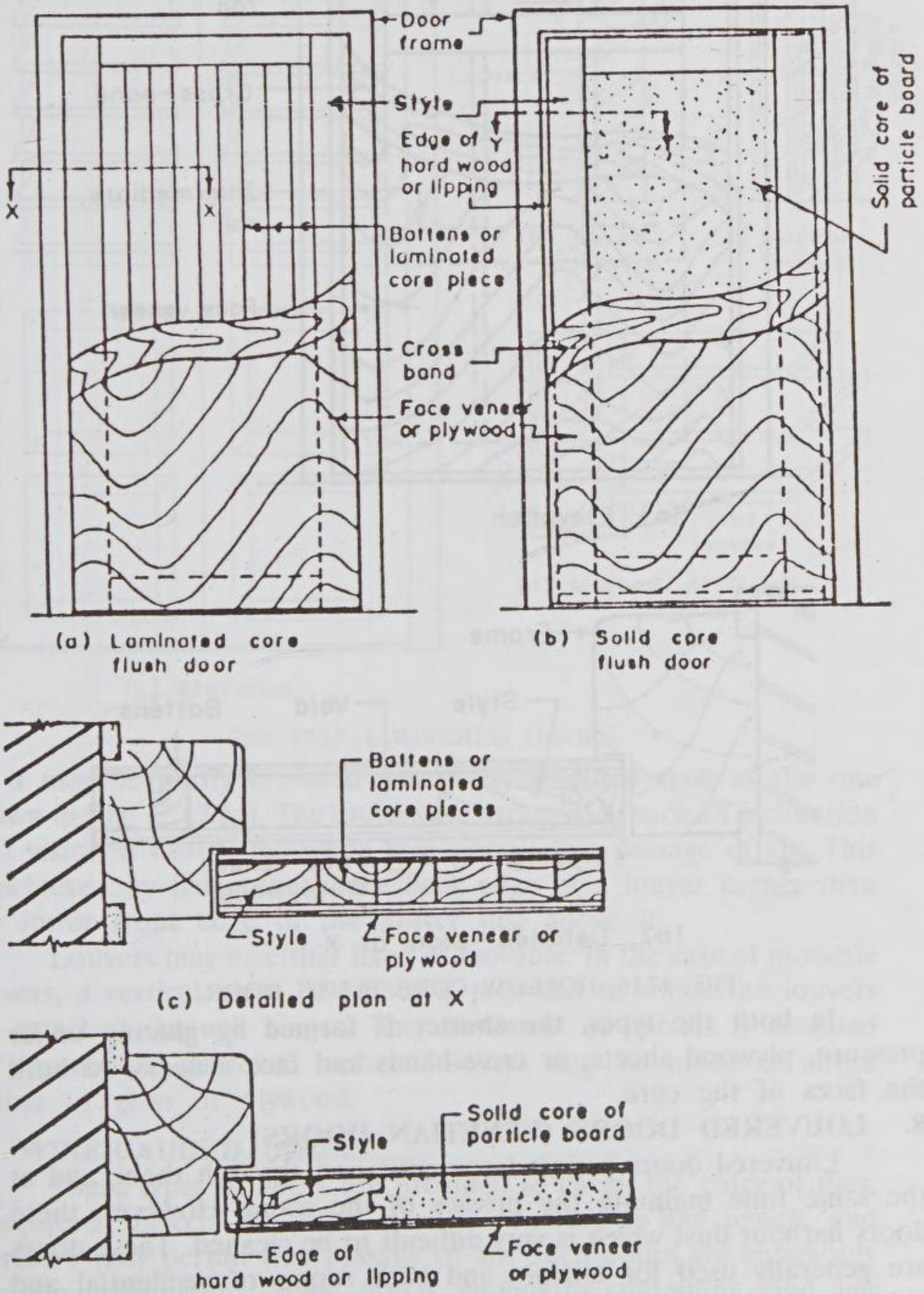
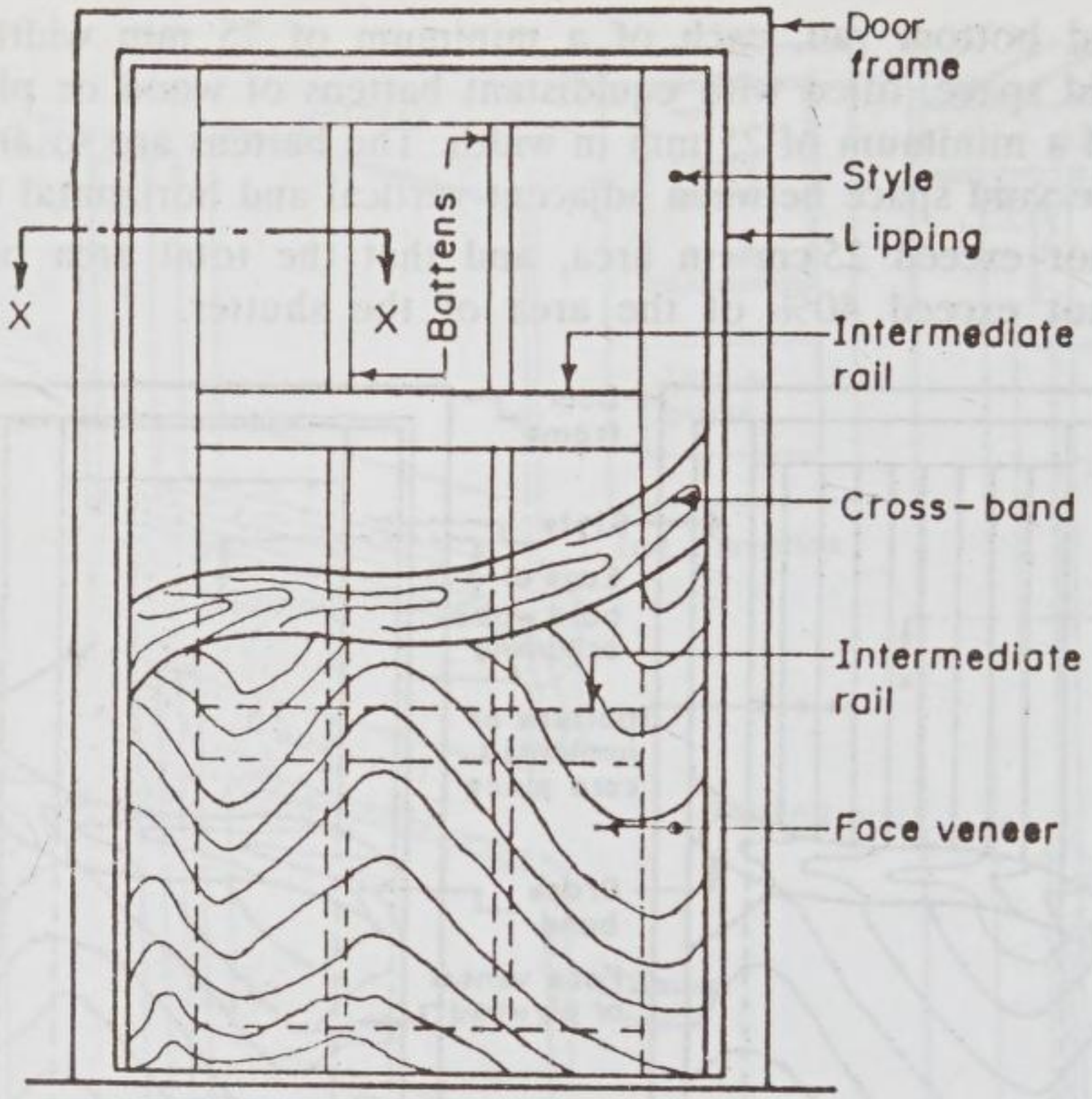
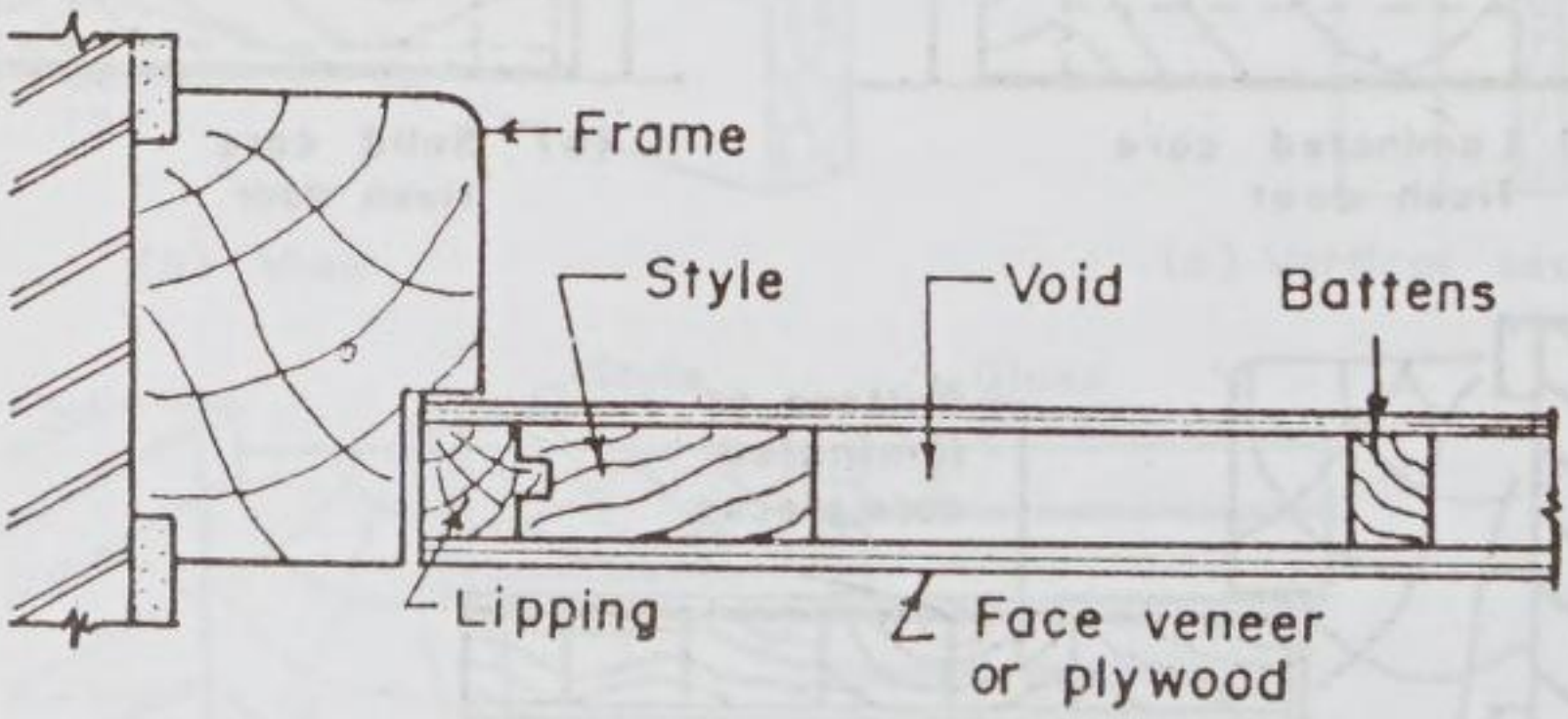


FIG. 17.15. SOLID CORE FLUSH DOORS.



(a) Elevation



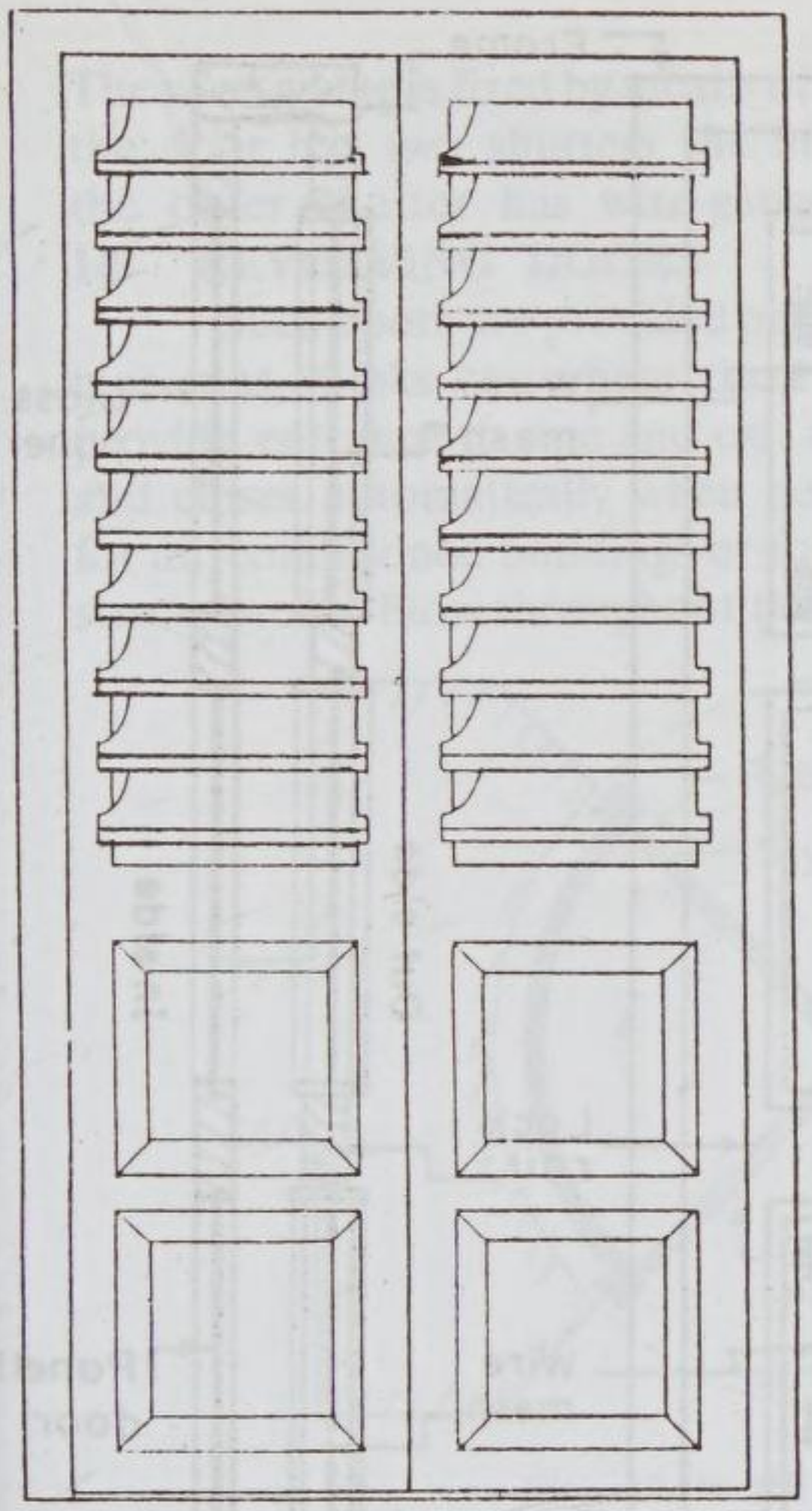
(b) Detailed plan at X

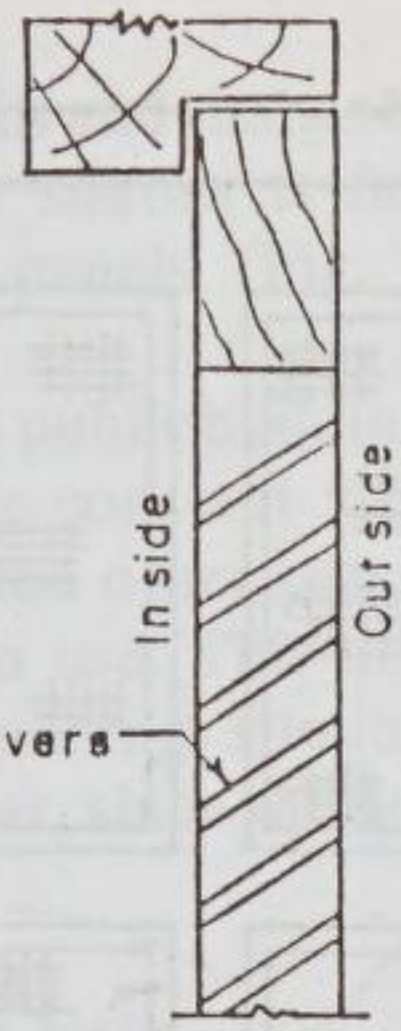
FIG. 17.16. HOLLOW CORE FLUSH DOOR.

In both the types, the shutter is formed by glueing under pressure, plywood sheets, or cross-bands and face veneers, to both the faces of the core.

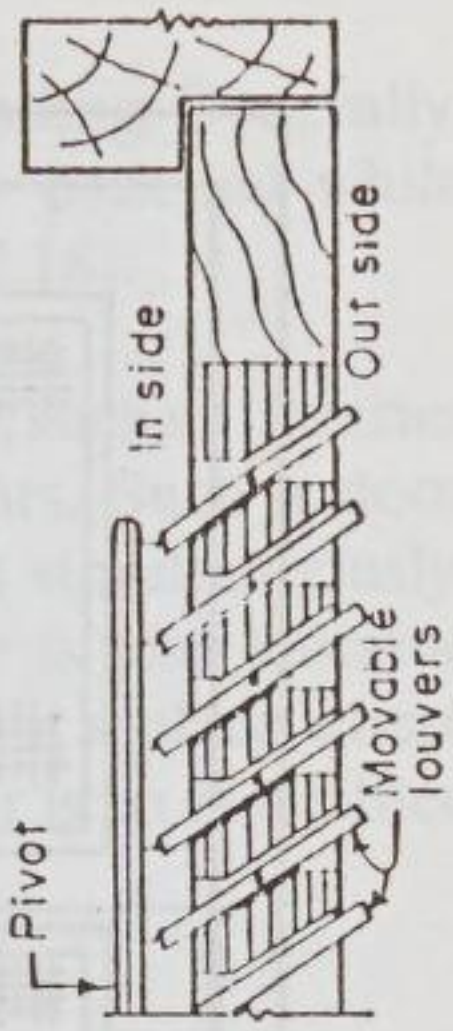
8. LOUVERED DOORS (VENETIAN DOORS)

Louvered doors permit free ventilation through them, and at the same time maintain the privacy of the room. However, these doors harbour dust which is very difficult to be cleaned. These doors are generally used for latrines and bath rooms of residential and public buildings. The door may either be louvered to its full height,

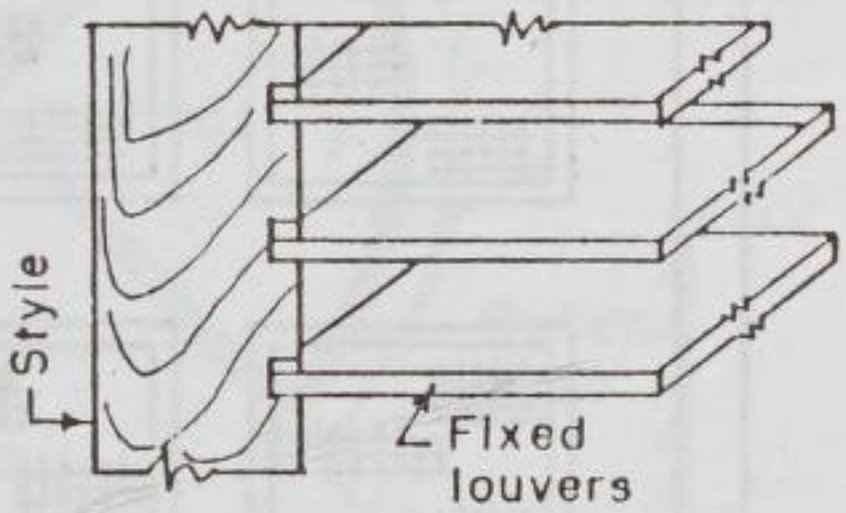




(b) Fixed louvers



(c) Movable louvers



(d) Method of fixing

FIG. 17.17. LOUVERED DOORS.

or it may be partly louvered and partly panelled such as the one shown in Fig. 17.17 (*a*). The louvers are arranged at such an inclination that vision is obstructed while they permit free passage of air. This is achieved by fixing the upper back edge of a louver higher than the lower front edge of the louver just above it.

Louvers may be either fixed or movable. In the case of movable louvers, a vertical piece of timber is provided to which the louvers are attached through hinges. The movement of louvers is actuated by the vertical piece of timber. Louvers may be made of either timber or glass or plywood.

9. WIRE-GAUGED DOORS

These types of doors are provided to check the entry of flies, mosquitoes, insects etc. Wire mesh is provided in the panels, and therefore they permit free passage of air. Such doors are commonly used for refreshment rooms, hotels, cupboards containing food and eatables, and sweet shops etc. The door is formed of a wooden

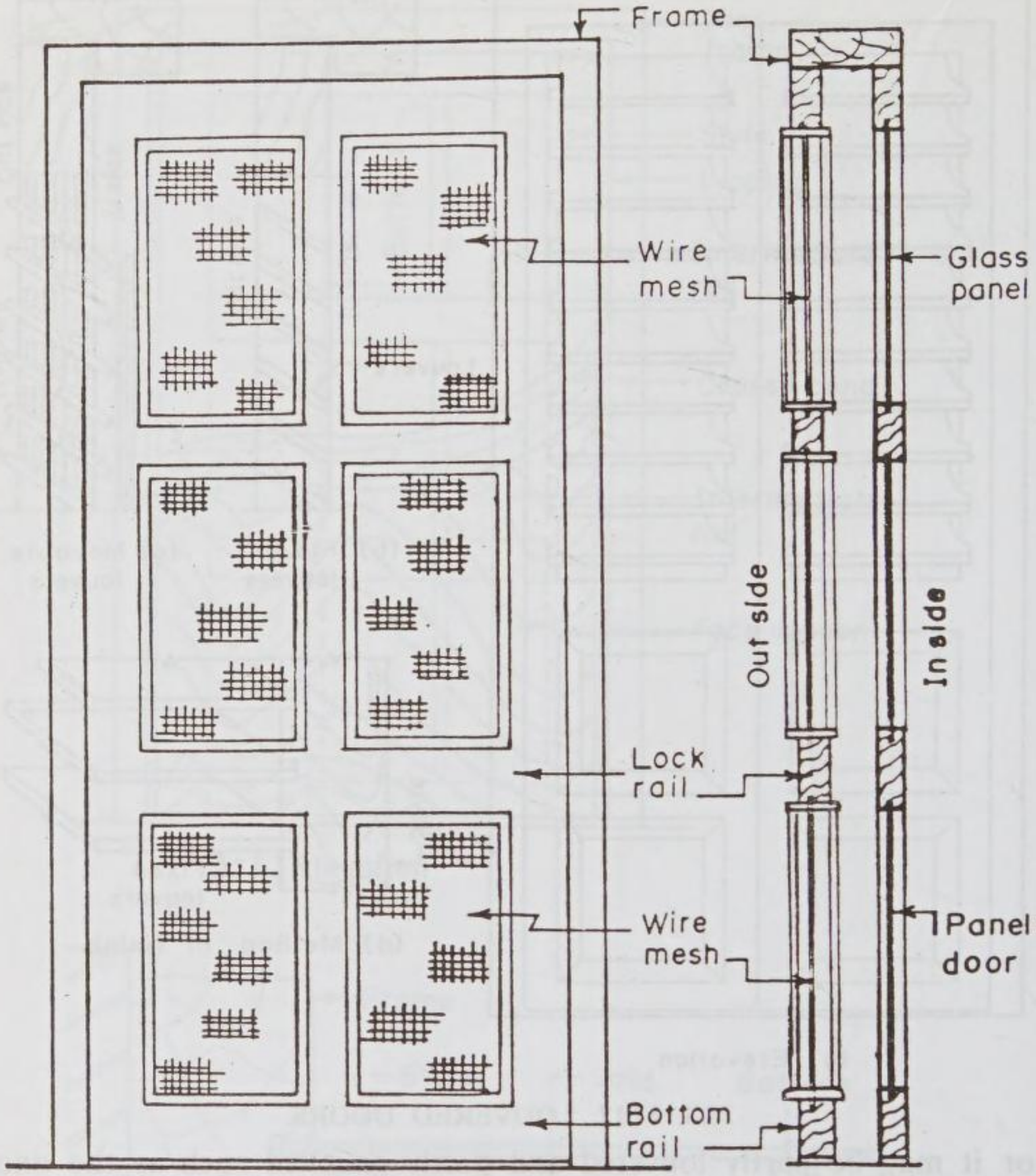


FIG. 17.18. WIRE-GAUGED DOOR

frame work consisting of vertical styles and horizontal rails, and the panel openings are fitted with fine mesh galvanized wire-gauge.

The wire-gauge is fixed by means of nails and timber beading. Generally, the door has two shutters the inner shutter is fully panelled while the outer shutter has wire-gauged panels (Fig. 17.18).

10. REVOLVING DOORS

Such doors are provided only in public buildings, such as libraries museums, banks etc. where there are constant visitors. Such a door provide entrance to one and exit to the other person simultaneously, and closes automatically when not in use. This door is also suitable for air-conditioned buildings or for buildings situated at a place where strong breeze blow throughout the year, since the door is so assembled

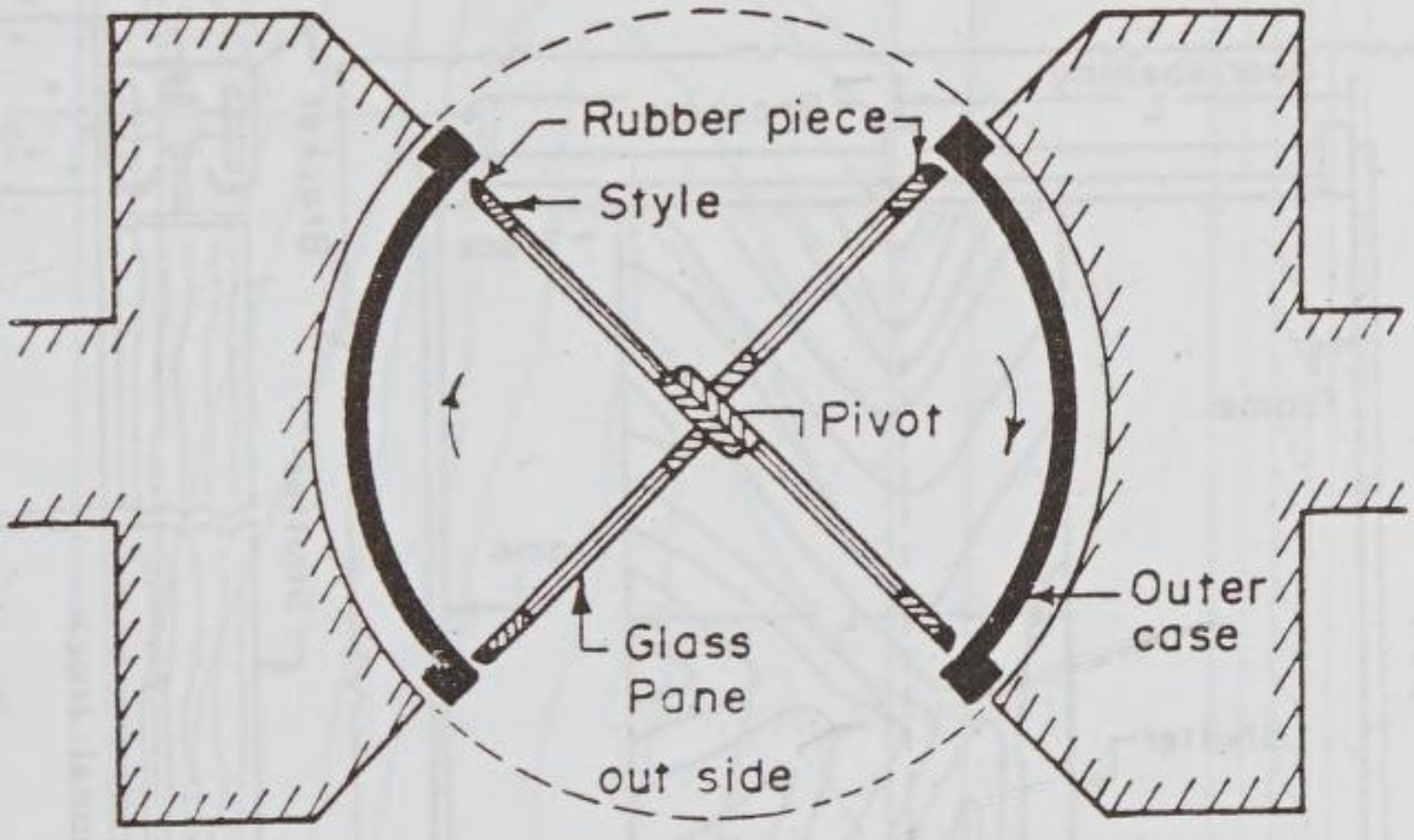


FIG. 17.19. REVOLVING DOOR.

that it excludes the wind drought. The door consists of a centrally placed mullion to which four radiating shutters are attached, as shown in Fig. 17.19. The mullion or vertical member is supported on ball bearings at the bottom, and has bush bearing at the top, so that its rotation is without any jerk, friction and noise. the shutters may be fully glazed, fully panelled or partly glazed and partly panelled. The shutters and the mullion are enclosed in a vestibule. Vertical rubber pieces are provided at the rubbing ends of shutters to prevent drought of air. The radiating shutters can be folded where traffic is more. The opening can also be closed.

11. SLIDING DOORS.

In such a door, the shutter slides on the sides with the help of runners and guide rails. The door may have one sliding shutter, two shutters or even three shutters, depending upon the size of the opening and the space available on sides for sliding. Fig. 17.20 (*a*) shows various types of sliding arrangements. Fig. 17.20 (*b*) shows the front view of a sliding door with single shutter, while Fig. 17.20 (*c*) shows its vertical section.

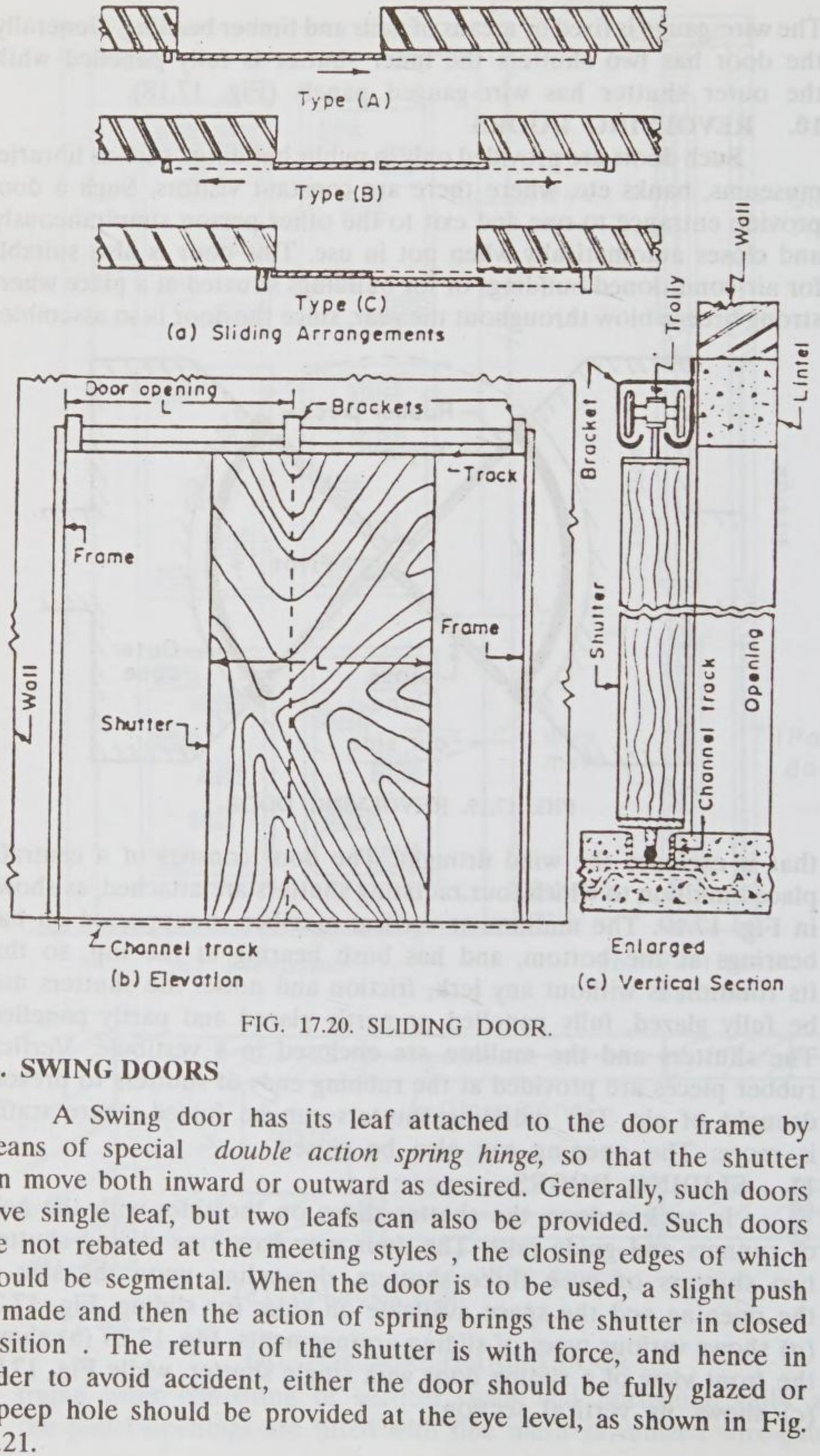
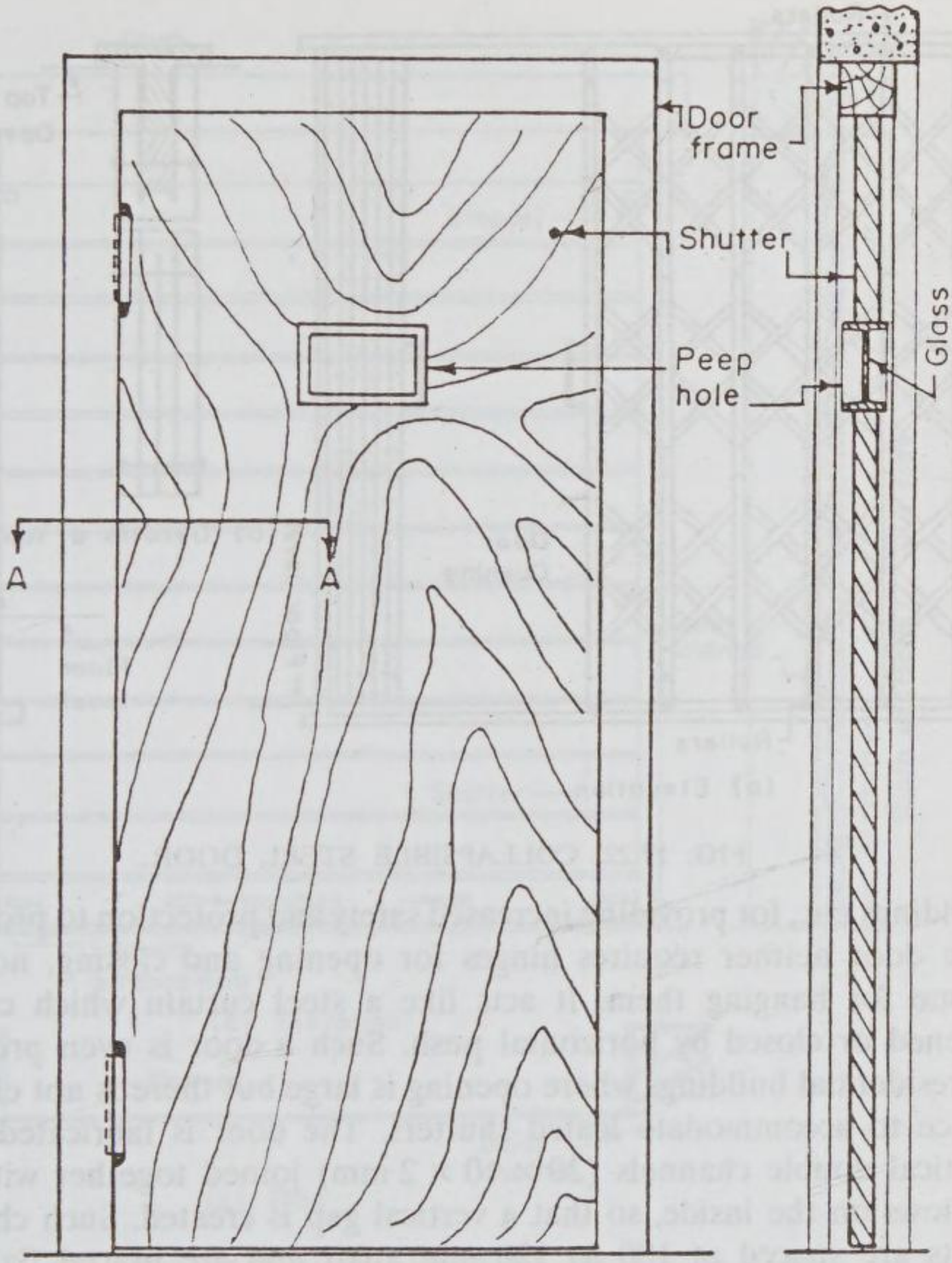


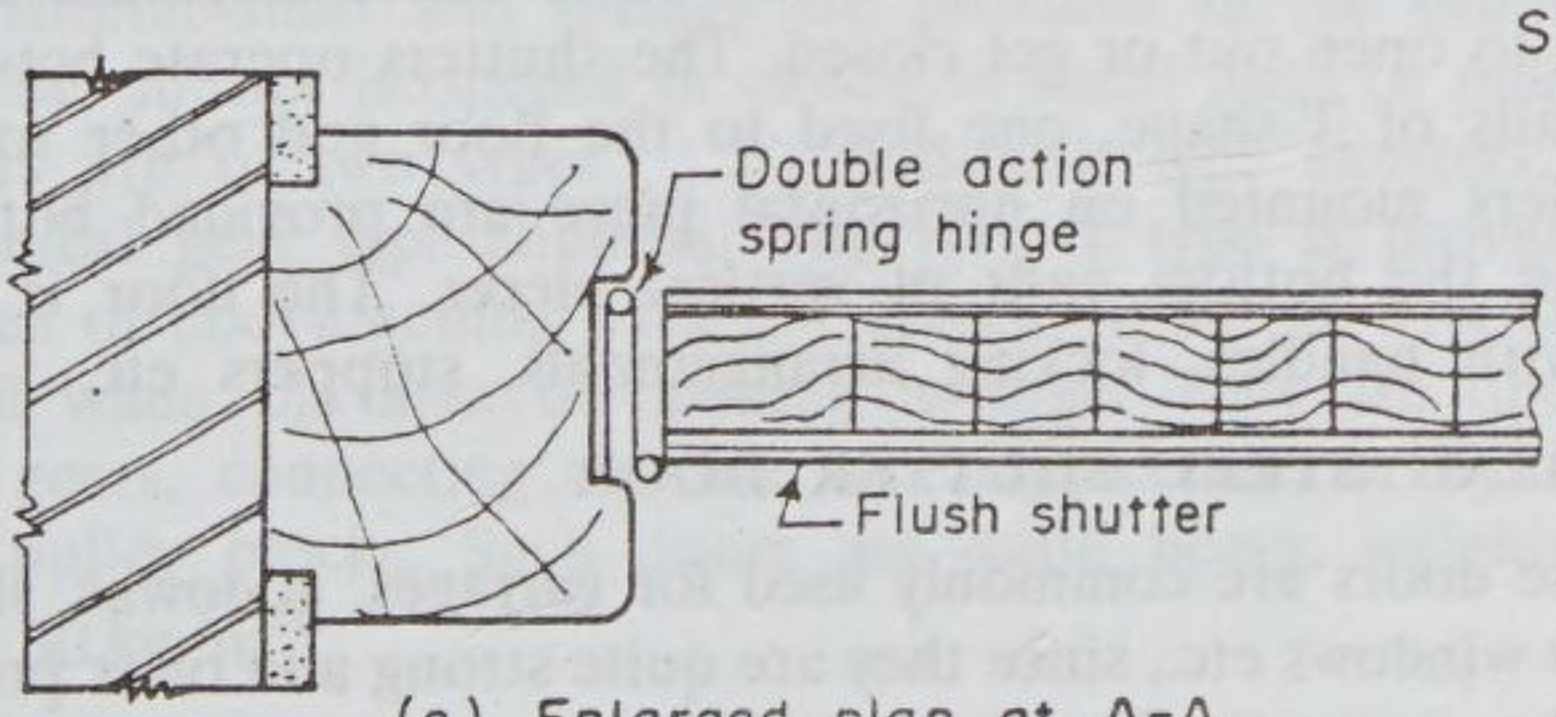
FIG. 17.20. SLIDING DOOR.

12. SWING DOORS

A Swing door has its leaf attached to the door frame by means of special *double action spring hinge,* so that the shutter can move both inward or outward as desired. Generally, such doors have single leaf, but two leafs can also be provided. Such doors are not rebated at the meeting styles , the closing edges of which should be segmental. When the door is to be used, a slight push is made and then the action of spring brings the shutter in closed position . The return of the shutter is with force, and hence in order to avoid accident, either the door should be fully glazed or a peep hole should be provided at the eye level, as shown in Fig. 17.21.



(a) Elevation (b) Vertical Section



(c) Enlarged plan at A-A

FIG. 17.21. SWING DOOR.

13. COLLAPSIBLE STEEL DOORS

Such doors are used in godowns, workshops, sheds, public

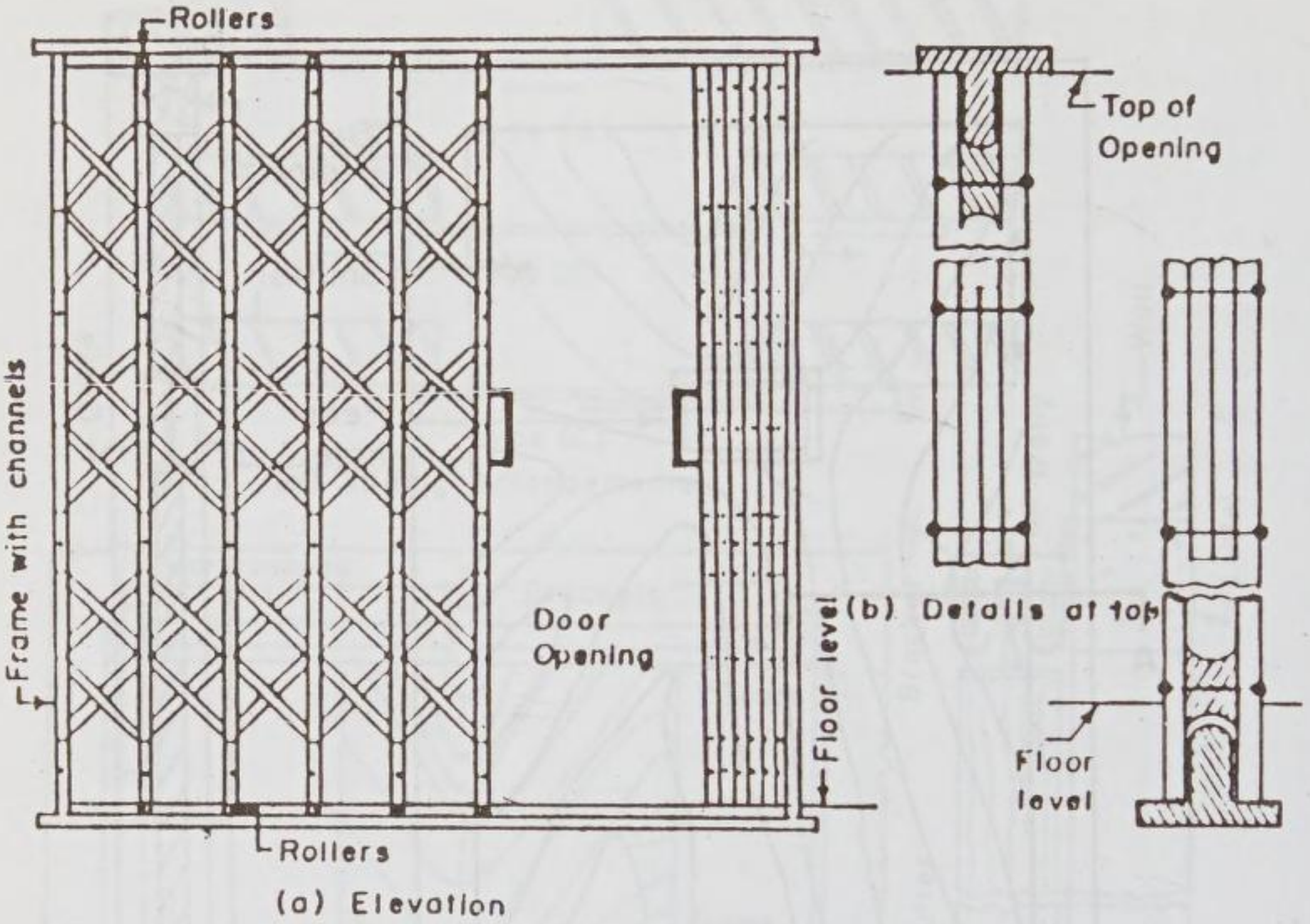


FIG. 17.22. COLLAPSIBLE STEEL DOOR..

buildings etc., for providing increased safety and protection to property. The door neither requires hinges for opening and closing, nor any frame for hanging them. It acts like a steel curtain which can be opened or closed by horizontal push. Such a door is even provided in residential buildings where opening is large but there is not enough space to accommodate leafed shutters. The door is fabricated from vertical double channels (20 × 10 × 2 mm) joined together with the hollows on the inside, so that a vertical gap is created. Such channel units are spaced at 100 to 120 mm apart and are braced flat iron diagonals 10 to 20 mm wide and 5 mm thick. These diagonals allow the shutter to open out or get closed. The shutters operate between two iron rails of T-shape, one fixed to the floor and other to the lintel. Rollers mounted on horizontal piece are provided both at the top and the bottom ends of vertical pieces. The door is also provided with handles, locking arrangements, stoppers etc.

14. ROLLING STEEL SHUTTER DOOR

These doors are commonly used for garrages, godowns, shops fronts show windows etc., since they are quite strong and offer proper safety to the property. The door consists of a frame, a drum and a shutter of thin steel plates (known as laths or slates), about 1 to 1.25 mm thick and inter locked together. The frame has steel guides on the sides in which the shutter moves, and then coils in the drum. The diameter of the drum varies from 200 to 300 mm.

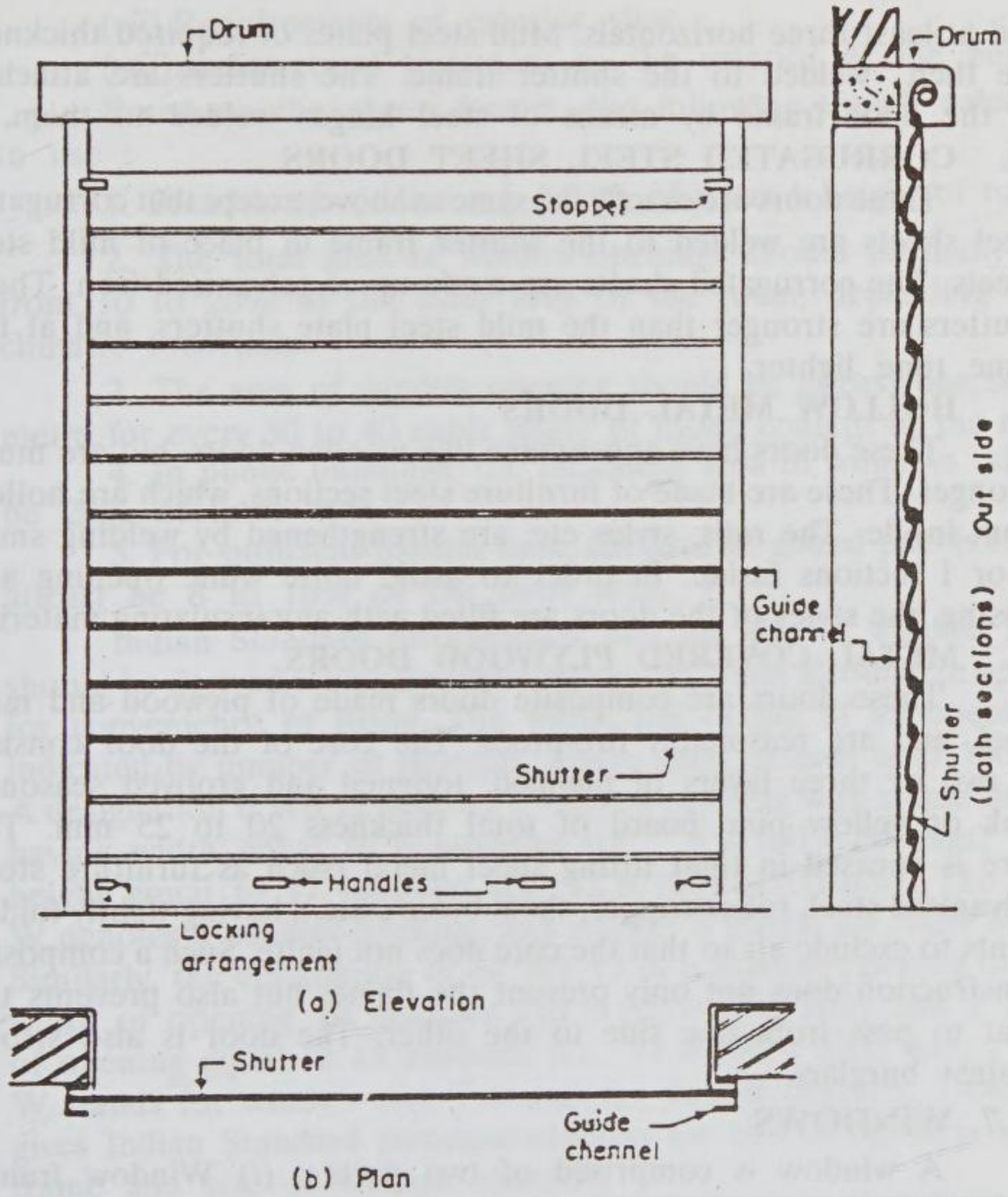


FIG. 17.23. ROLLING STEEL SHUTTERS.

A horizontal shaft and springs are provided in the drum, due to which the shutter is opened or closed by small push or pull. Rolling shutters are of two types : (*i*) pull-push type shutters, and (*ii*) mechanical gear type shutters. The former type is provided when the area of door-opening does not exceed 10 Sq. m. The latter type is used when the area of opening is large. They are operated by worm gears, connecting rod and winding handle, or by means of chain pulley blocks. Such doors are quite heavy, weighing about 25 to 30 kg/m^2.

15. MILD STEEL SHEET DOORS

These doors are provided for garrages, godowns, workshops etc., and are quite strong. The door consists of a door frame made of angle or T-sections. The door has generally two shutters. Each shutter is made up of frame of angle of iron, having two verticals

and at least three horizontals. Mild steel plates of required thickness are then welded to the shutter frame. The shutters are attached to the door frame by means of steel hinges welded to them.

16. CORRUGATED STEEL SHEET DOORS

These doors are exactly the same as above, except that corrugated steel sheets are welded to the shutter frame in place of mild steel sheets. The corrugated sheets are made up of galvanised iron. These shutters are stronger than the mild steel plate shutters, and at the same time lighter.

17. HOLLOW METAL DOORS

These doors have appearance like wooden doors, but are much stronger. These are made of furniture steel sections, which are hollow from inside. The rails, styles etc. are strengthened by welding small T or I sections inside. In order to avoid noise while opening and closing, the styles of the doors are filled with any insulating material.

18. METAL COVERED PLYWOOD DOORS

These doors are composite doors made of plywood and mild steel, and are reasonably fire-proof. The core of the door consists of two or three layers of planned, tongued and grooved seasoned teak or yellow pine board of total thickness 20 to 25 mm. The core is encased in tight fitting sheet metal (such as furniture steel, galvanised steel, roller copper, sheet bronze etc.), having tightly folded joints to exclude air so that the core does not ignite. Such a composite construction does not only prevent the flames but also prevents the heat to pass from one side to the other. The door is also strong against burglars.

17.7. WINDOWS

A window is comprised of two parts : (*i*) Window frame, and (*ii*) Sashes or shutter frame. Window frames are fixed to the opening in the wall, by means of suitable hold fasts. The sashes or shutter frames are fixed to the window frames by means of suitable hinges. The window frame has *sill* at the bottom, unlike doors.

The function of the window is to admit light and air to the room and to give a view to the outside. It should also provide insulation against heat loss and, in some cases, against sound.

The selection of size, shape, location and number of windows in a room depends upon the following factors :

(*i*) Size of the room.
(*ii*) Location of the room.
(*iii*) Utility of the room.
(*iv*) Direction of the wall.
(*v*) Direction of wind.
(*vi*) Climatic conditions such as humidity, temperature etc.

(*vii*) Requirements of exterior view.

(*viii*) Architectural treatment to the exterior of the building.

Based on the above factors, the following thumb rules are in use :

1. Breadth of window $= \frac{1}{8}$ (width of room + height of room).

2. The total area of window-openings should normally vary from 10 to 20% of the floor area of the room, depending upon climatic conditions.

3. The area of window-opening should be atleast one square metre for every 30 to 40 cubic metre of inside content of the room.

4. In public buildings, the minimum area of windows should be 20% of floor area.

5. For sufficient natural light, the area of glazed panels should atleast be 8 to 10% of the floor area.

Indian Standard recommends that the size of window frame should be derived after allowing a margin of 5 mm alround an *opening* for convenience of fixing. The width and height of an opening is indicated by number of modules, where each module is of 100 mm. A designation 6 WS 12 denotes a window-opening with single shutter, having width equal to 6 modules (*i.e.* $6 \times 100 = 600$ mm) and height equal to 12 modules (*i.e.* $12 \times 100 = 1200$ mm). The letter W denotes a window-opening, and a letter S stands for *single shutter.* Similarly, the designation of 10 WT 13 of a window-opening denotes width of opening 10 modules ($10 \times 100 = 1000$ mm) and a height of opening equal to 13 modules (*i.e.*, $13 \times 100 = 1300$ mm) ; letter W stands for window and T stands for double shutters. Table 17.2 gives Indian Standard recommendations for size of opening, size of frame and size of window shutters.

TABLE 17.2. RECOMMENDED DIMENSIONS FOR WINDOWS

S.No. (1)	*Designation* (2)	*Size of opening (mm)* (3)	*Size of window frame (mm)* (4)	*Size of window shutter (mm)* (5)
1.	6 WS 12	600×1200	590×1190	500×1100
2.	10 WT 12	1000×1200	990×1190	460×1100
3.	12 WT 12	1200×1200	1190×1190	560×1100
4.	6 WS 13	600×1300	590×1290	500×1200
5.	10 WT 13	1000×1300	990×1290	460×1200
6.	12 WT 13	1200×1300	1190×1290	560×1200

17.8. TYPES OF WINDOWS

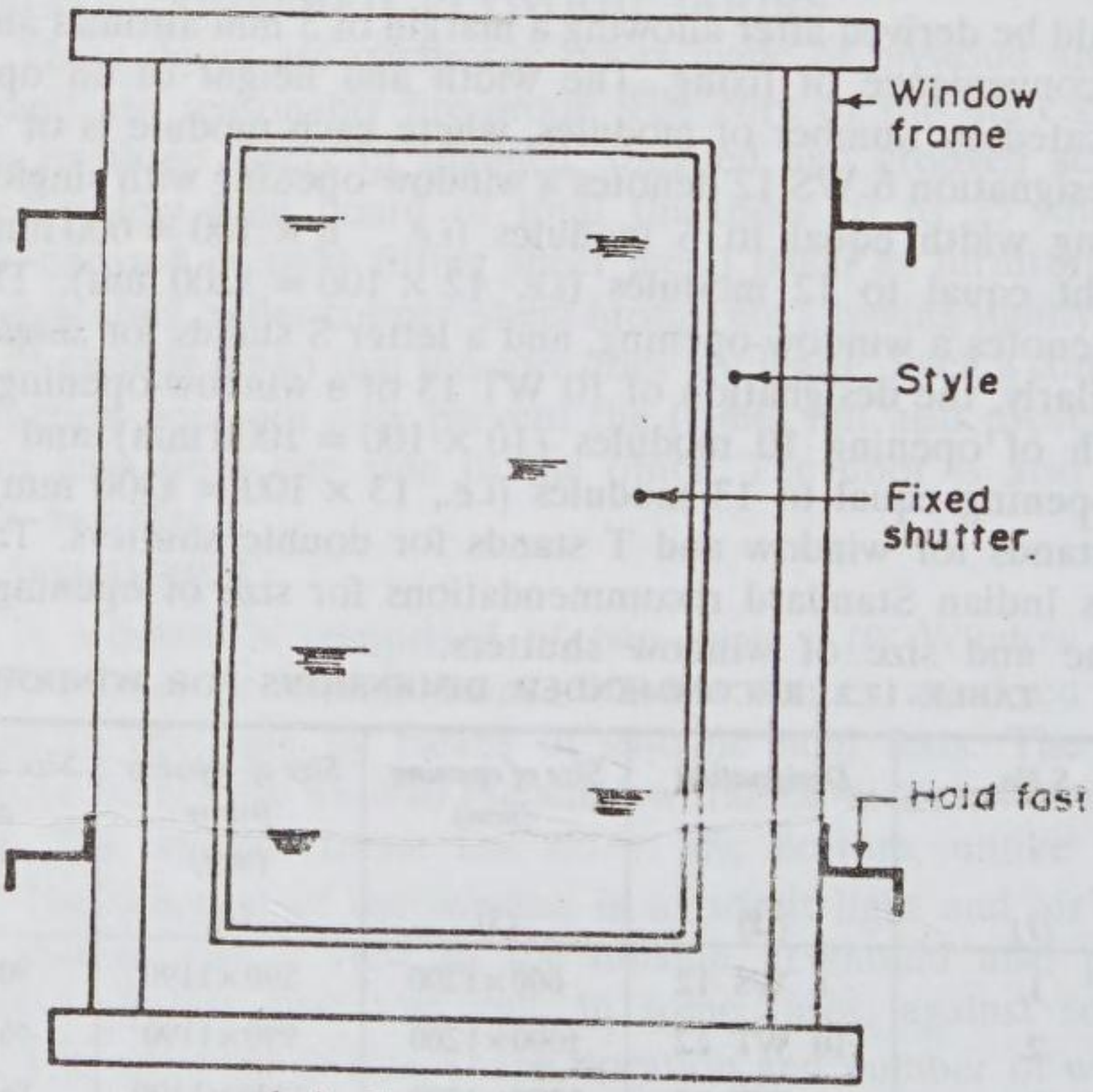
Windows are classified as follows, based on the nature of operational movements of shutters, materials used for construction,

manner of fixing and their location.

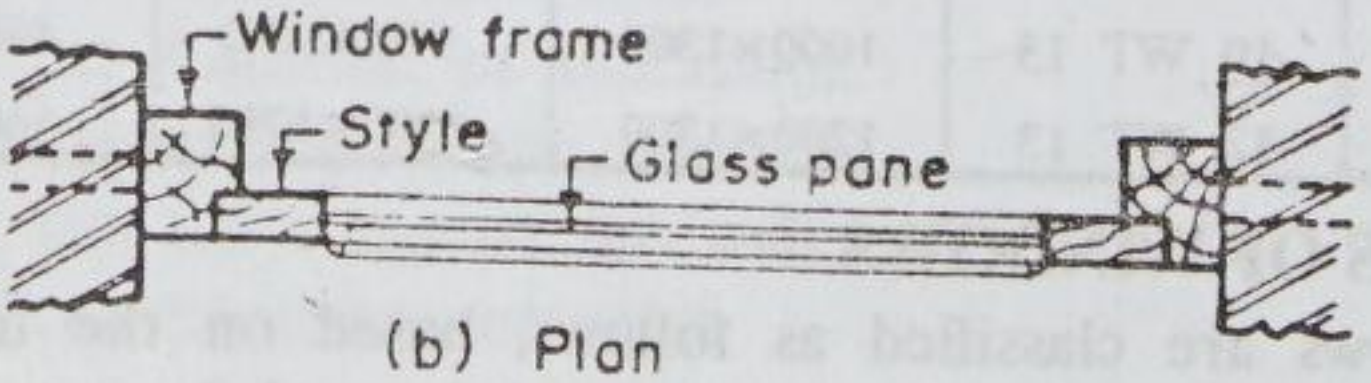
1. Fixed Windows.
2. Pivoted windows.
3. Double hung windows.
4. Sliding windows.
5. Casement windows.
6. Sash windows.
7. Louvered windows.
8. Metal windows.
9. Bay windows.
10. Clerestorey windows.
11. Corner windows.
12. Dormer windows.
13. Cable windows.
14. Lantern windows.
15. Skylights.
16. Ventilators.
17. Combined windows and ventilators.

1. FIXED WINDOWS

These windows are provided for the sole purpose of admitting light and/or providing vision in the room. The window consists of



(a) Elevation



(b) Plan

FIG. 17.24. FIXED WINDOWS.

a window frame to which shutters are fixed. No rebates are provided to the frame. The shutters are fully glazed.

2. PIVOTED WINDOWS

In these windows, the shutters are allowed to swing round pivots fixed to the window frame. The window frame has no rebates. The frame of the window shutter is similar to that of an encasement window. The shutter can swing or rotate either horizontally, or vertically (Fig. 17.25).

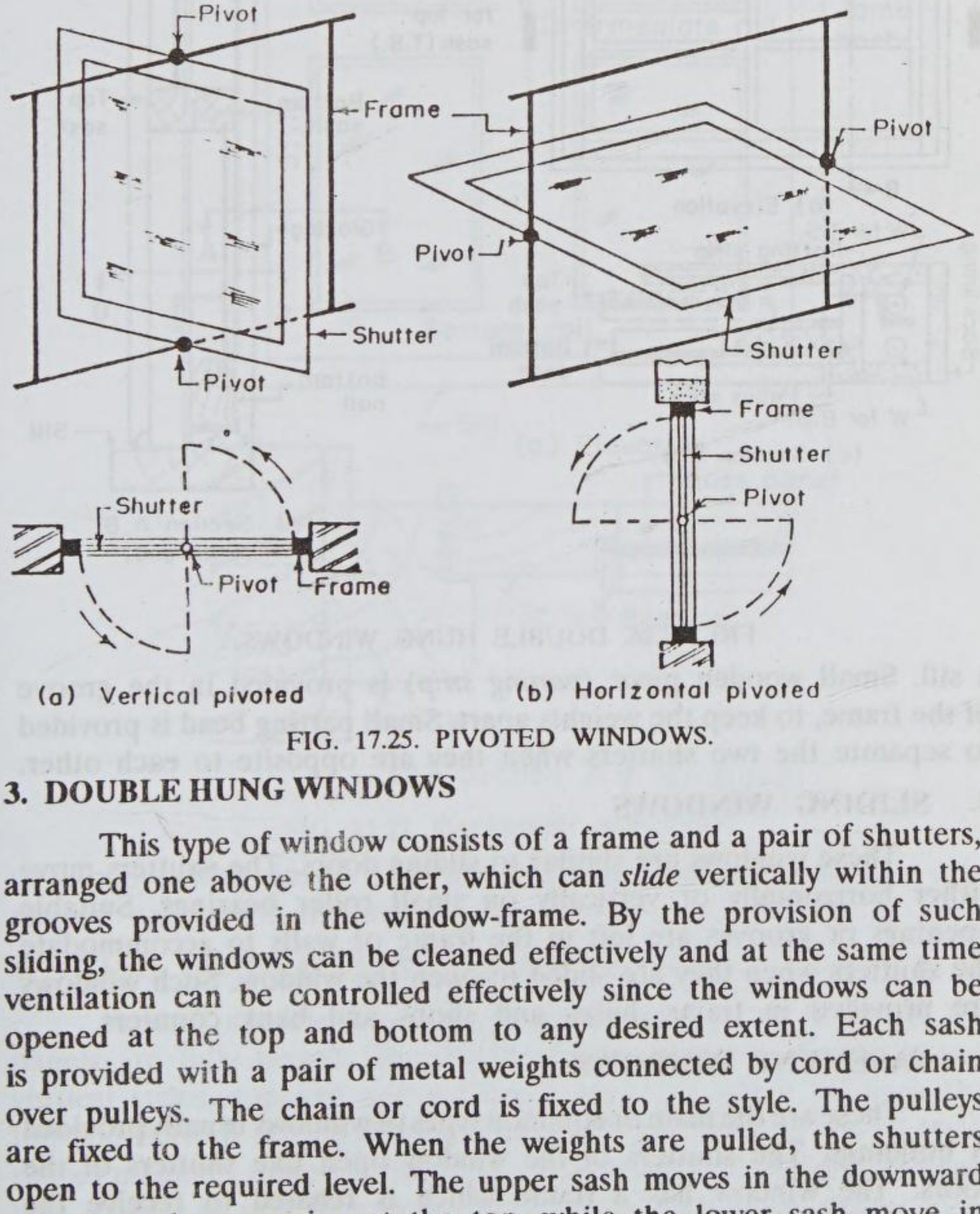


FIG. 17.25. PIVOTED WINDOWS.

3. DOUBLE HUNG WINDOWS

This type of window consists of a frame and a pair of shutters, arranged one above the other, which can *slide* vertically within the grooves provided in the window-frame. By the provision of such sliding, the windows can be cleaned effectively and at the same time ventilation can be controlled effectively since the windows can be opened at the top and bottom to any desired extent. Each sash is provided with a pair of metal weights connected by cord or chain over pulleys. The chain or cord is fixed to the style. The pulleys are fixed to the frame. When the weights are pulled, the shutters open to the required level. The upper sash moves in the downward direction, thus opening at the top, while the lower sash move in the upward direction thus opening at the bottom. Special frame, called *boxed frame or cased frame* is provided, consisting of two vertical members (spaced apart to provide the groove) a head and

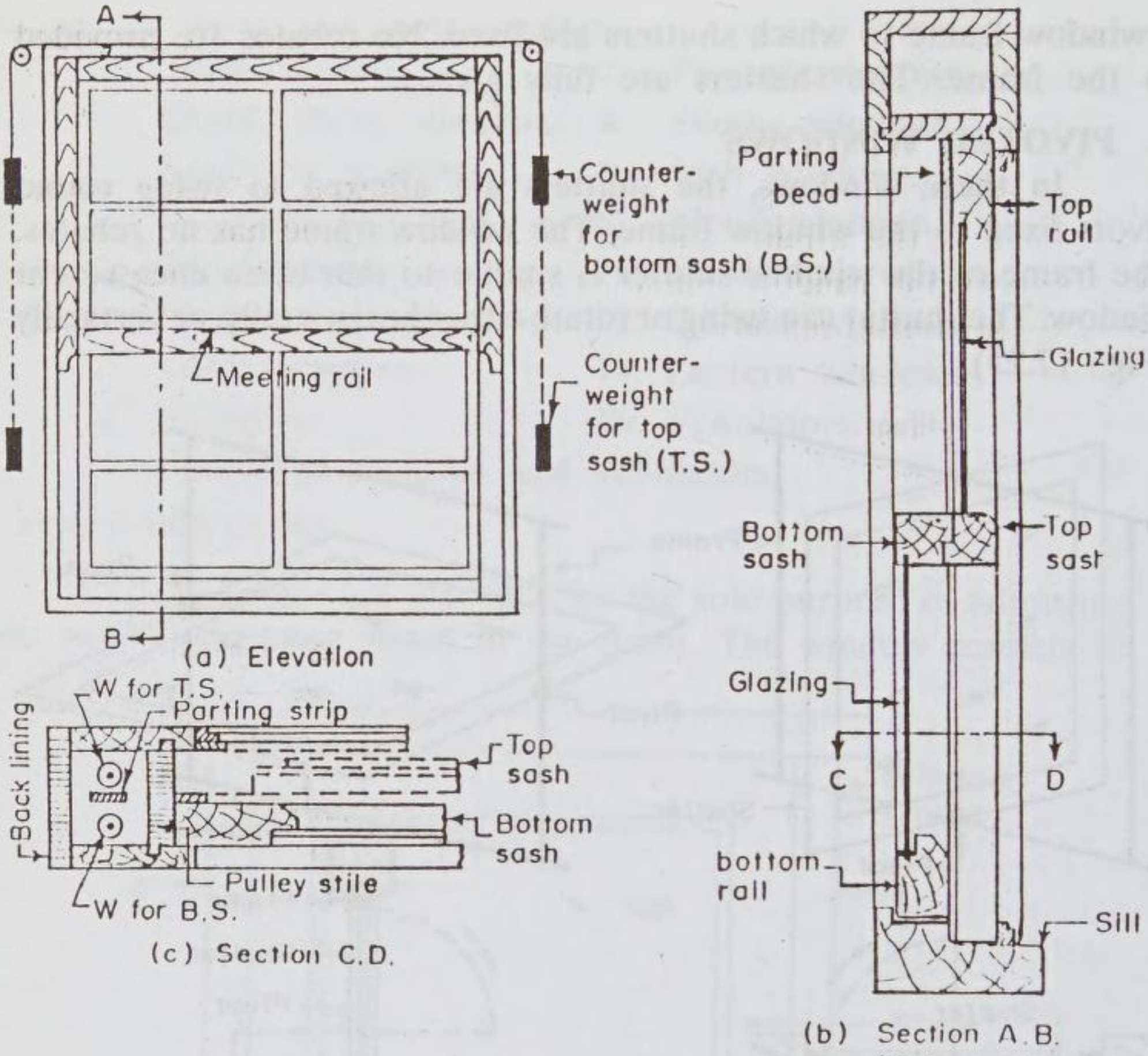


FIG. 17.26. DOUBLE HUNG WINDOWS.

a sill. Small wooden piece (*parting strip*) is provided in the groove of the frame, to keep the weights apart. Small parting bead is provided to separate the two shutters when they are opposite to each other.

4. SLIDING WINDOWS

These windows are similar to sliding doors. The shutters move either horizontally or vertically on small roller bearings. Suitable openings or grooves are left in the frame or walls to accommodate the shutters when they are slided to open the window. Such windows are provided in trains, buses and shops and bank counters.

5. CASEMENT WINDOWS

These are the main or common types of windows usually provided in buildings. The shutters of the window open like shutters of the doors. The window has a frame which is rebated to receive the shutters. The shutters consists of styles, top rails, bottom rails, and intermediate rails, thus dividing it into panels. The panels may either be glazed, or unglazed, or partly glazed and partly unglazed. In case

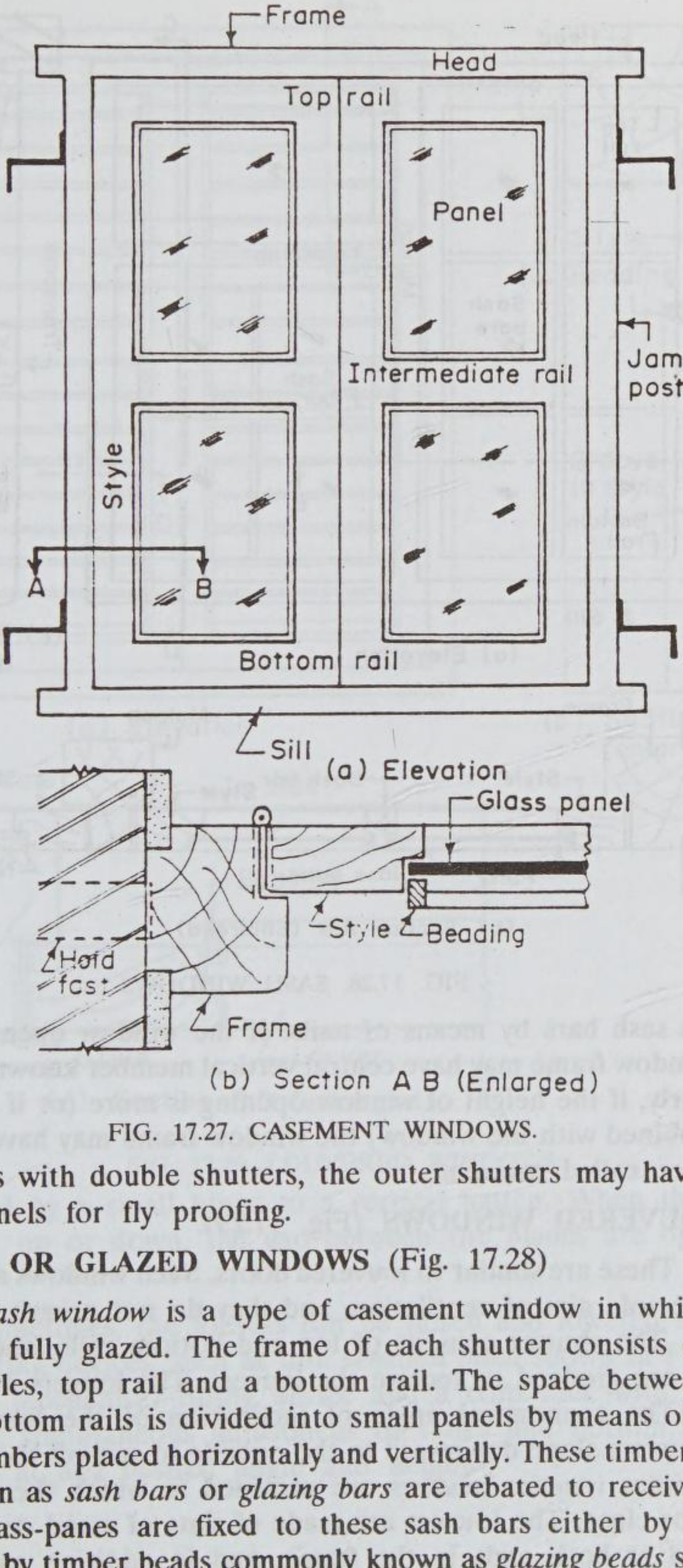


FIG. 17.27. CASEMENT WINDOWS.

of windows with double shutters, the outer shutters may have wire gauged panels for fly proofing.

6. SASH OR GLAZED WINDOWS (Fig. 17.28)

A *sash window* is a type of casement window in which the panels are fully glazed. The frame of each shutter consists of two vertical styles, top rail and a bottom rail. The space between the top and bottom rails is divided into small panels by means of small timber members placed horizontally and vertically. These timber members, known as *sash bars* or *glazing bars* are rebated to receive glass panels. Glass-panes are fixed to these sash bars either by means of putty or by timber beads commonly known as *glazing beads* secured

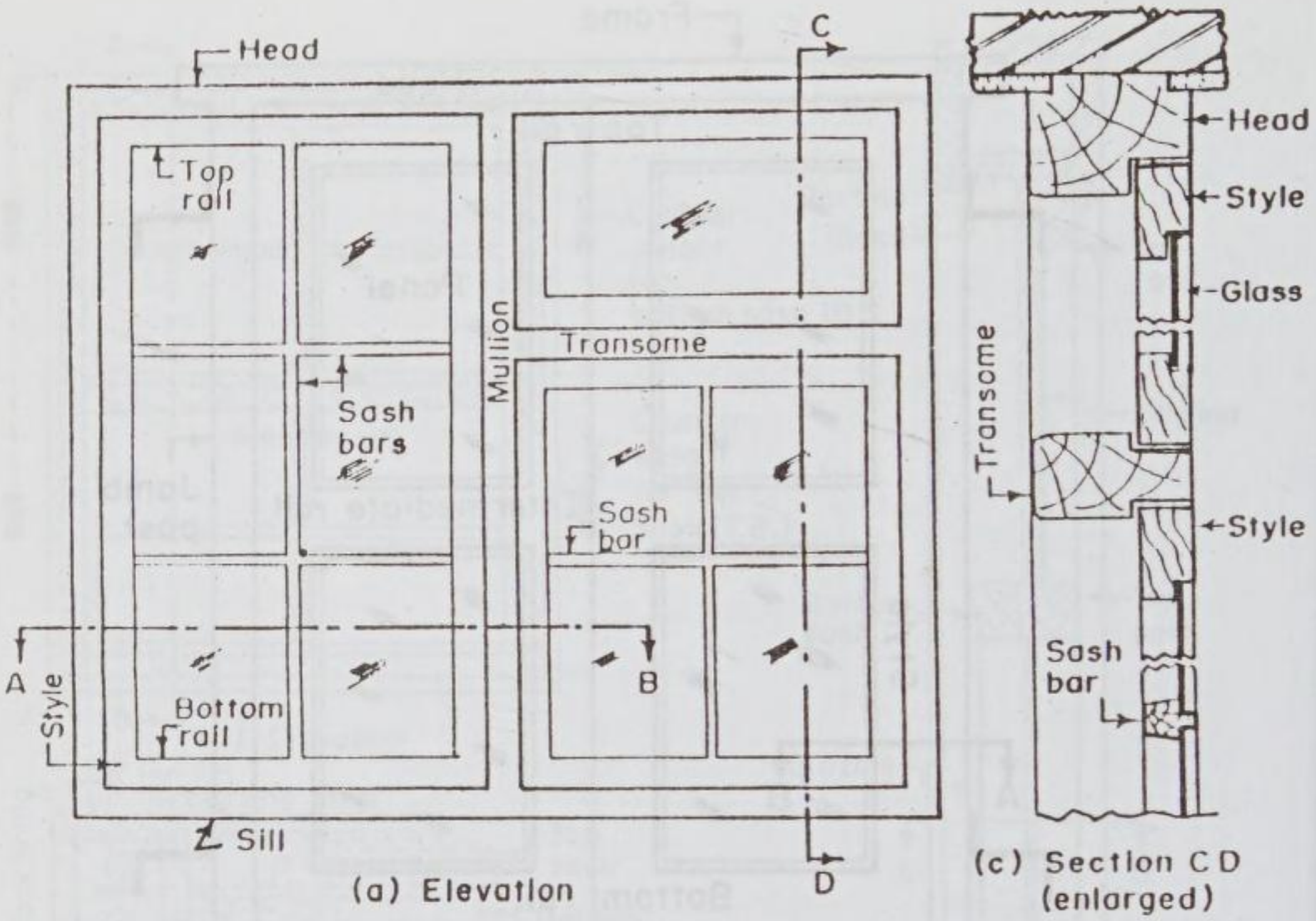
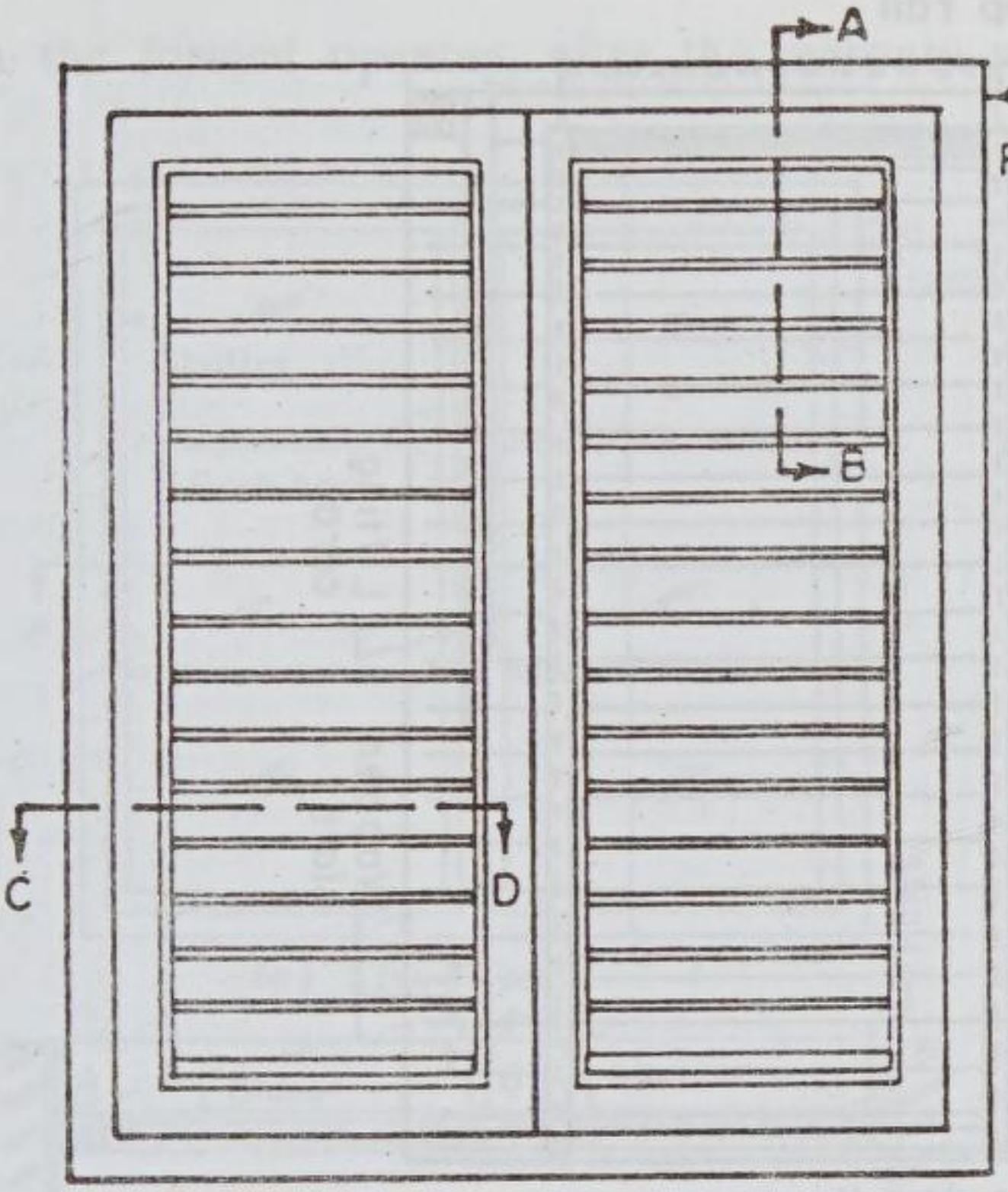


FIG. 17.28. SASH WINDOWS.

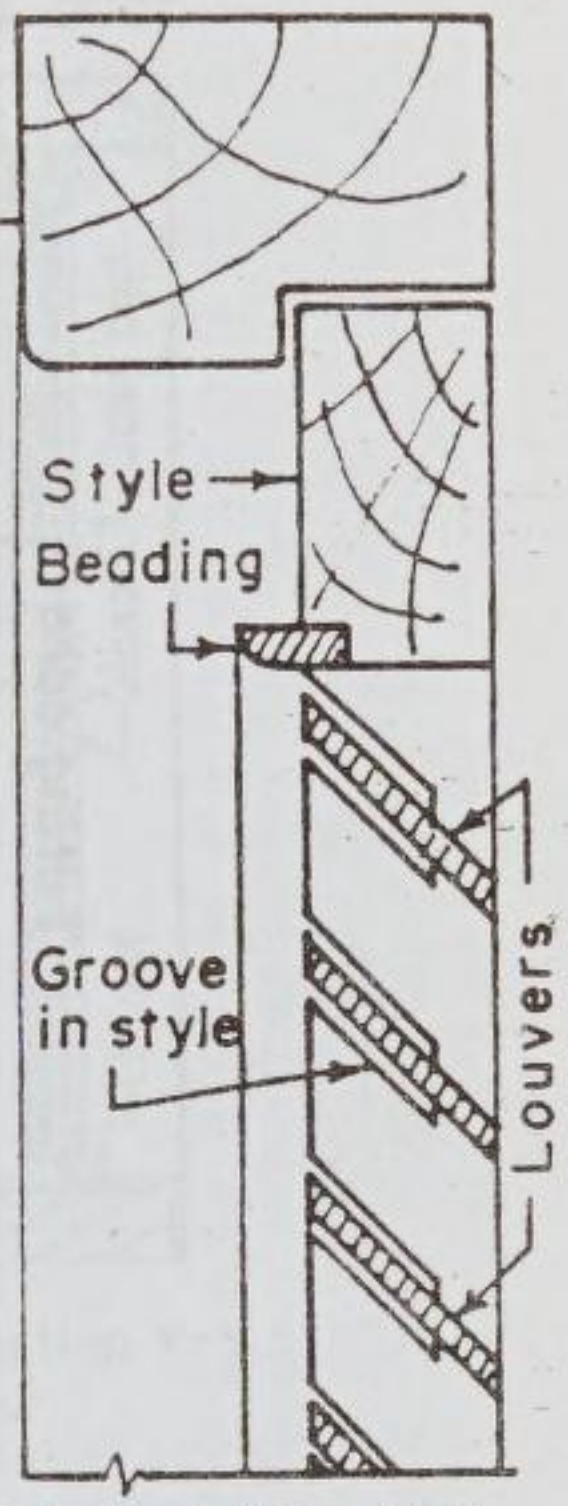
to the sash bars by means of nails. If the window opening is wide, the window frame may have central vertical member known as *mullion*. Similarly, if the height of window opening is more (or if a ventilator is combined with the window) the window frame may have horizontal member called *transome*.

7. LOUVERED WINDOWS (Fig. 17.29).

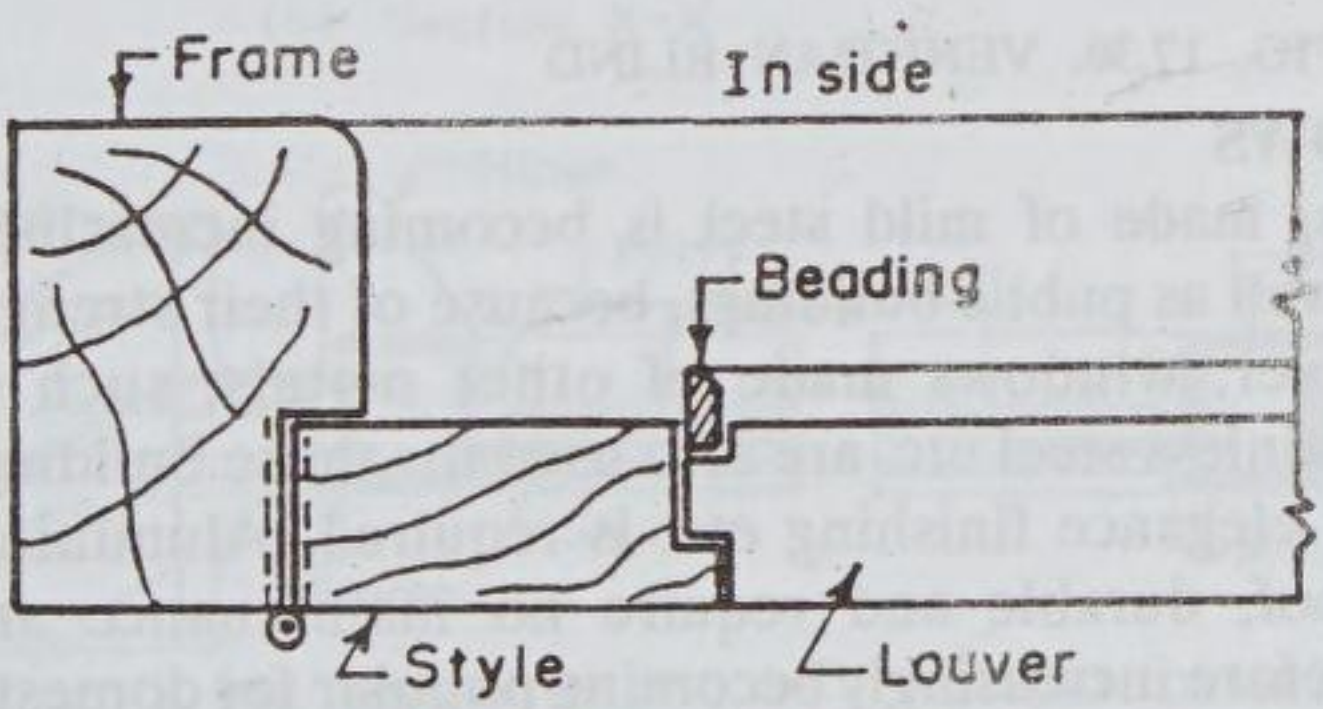
These are similar to louvered doors. Such windows are provided for the sole aim of ventilation, and they do not permit any outside vision. The shutter consists of top and bottom rails, and two styles which are grooved to receive the louvers. The louvers are generally fixed. The economical angle of inclination of the louvers is 45°. The louvers slope downward to the outside to run-off the rain water. Sometimes *venetian shutters* are provided in which the louvers can open or close. The louvers are made of slats of wood or aluminium, pivoted at both ends in the frame, and in addition each blade is



(a) Elevation



(b) Section A B
(enlarged)



(c) Section C D (enlarged)

FIG. 17.29. LOUVERED WINDOWS.

connected by a small hinge to a vertical batten. When the batten is pulled up or down, the gap between the blades are opened or closed.

Sometimes, the louvers can be raised and lowered by means of operating devices, such as in a *venetian blind* shown in Fig. 17.30. It has a mechanical tilting device and a cord lock with the help of which simultaneous adjustment of slates and bottom rails can be done at any desired angle and height.

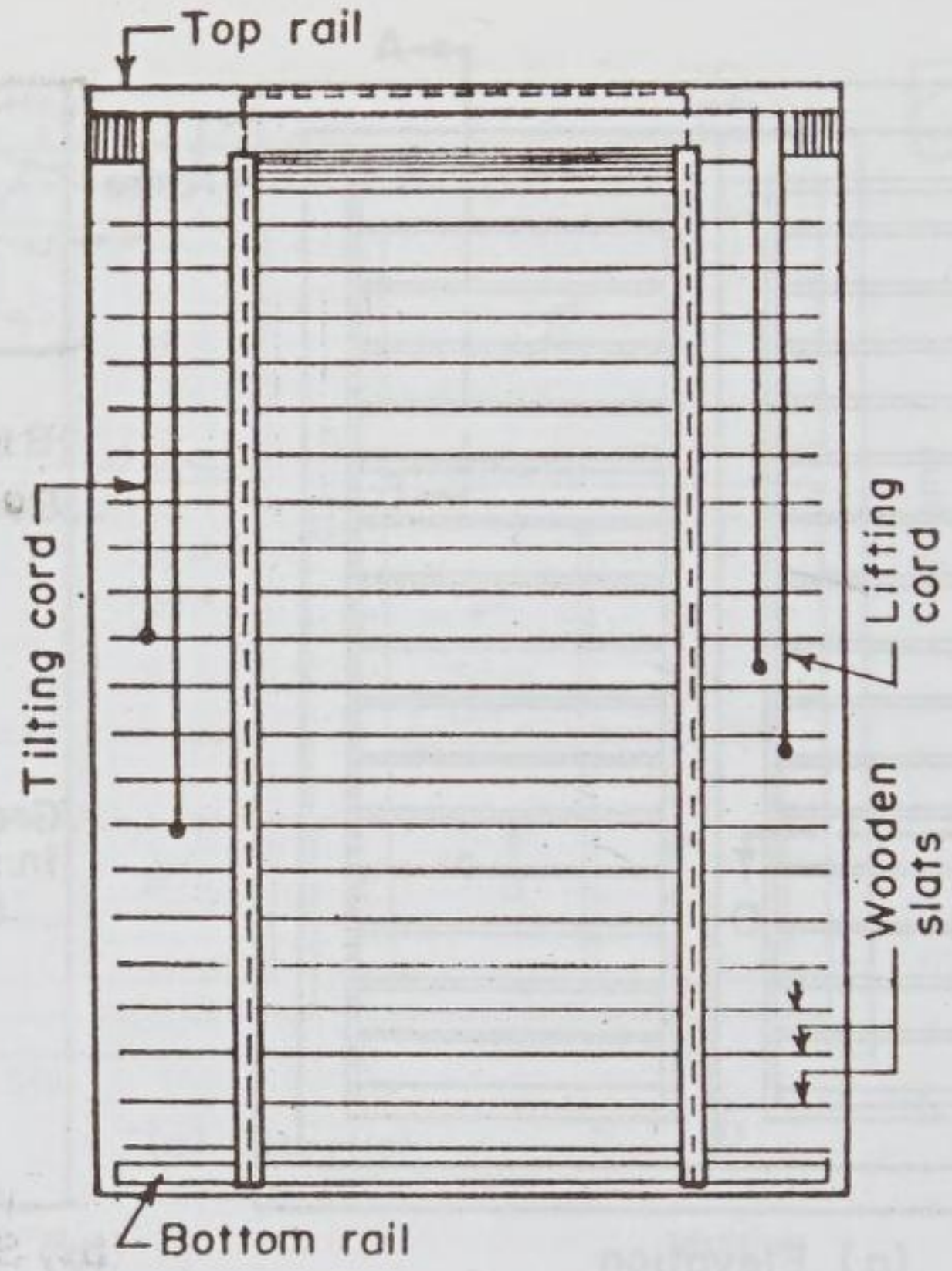


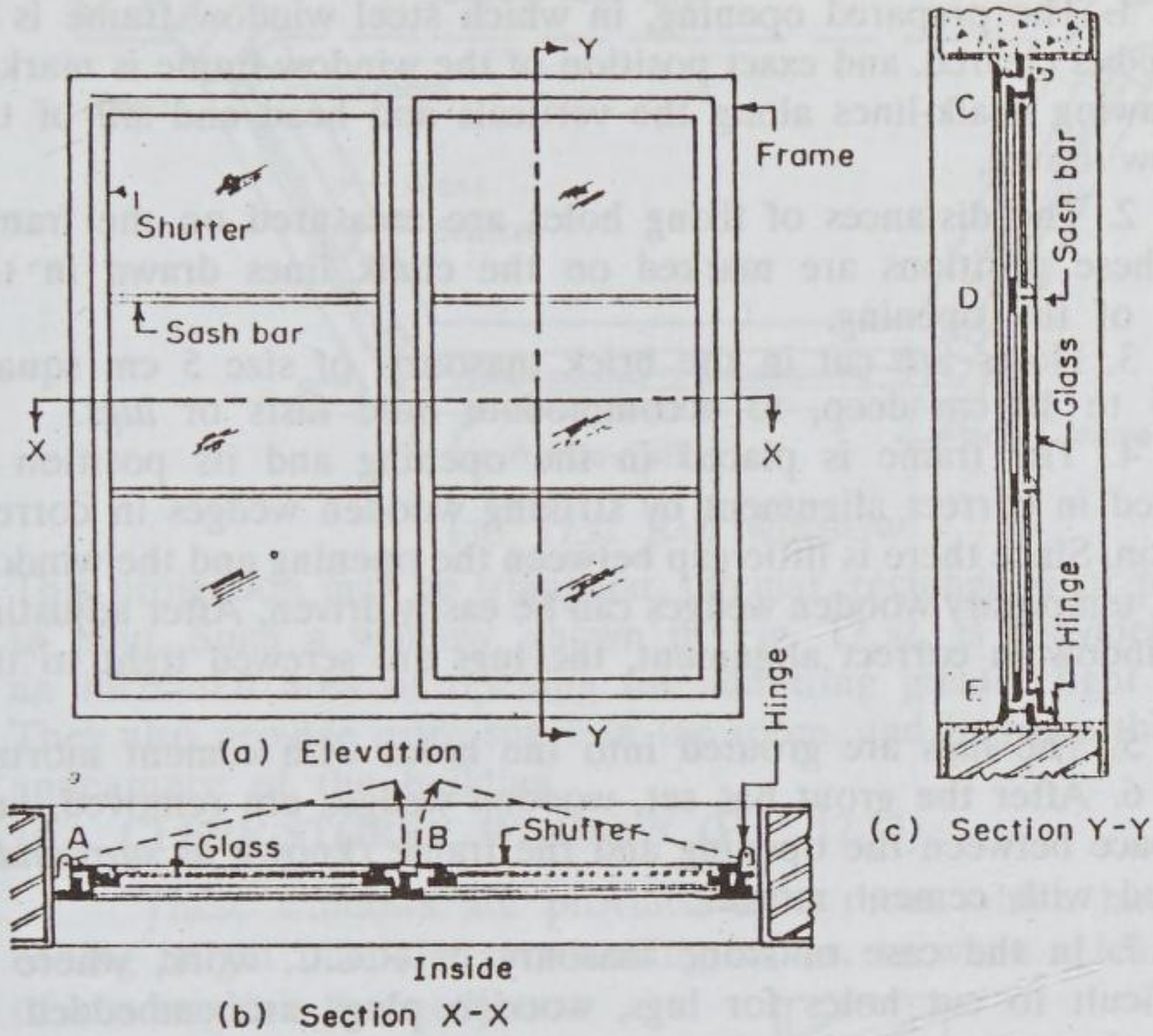
FIG. 17.30. VENETIAN BLIND

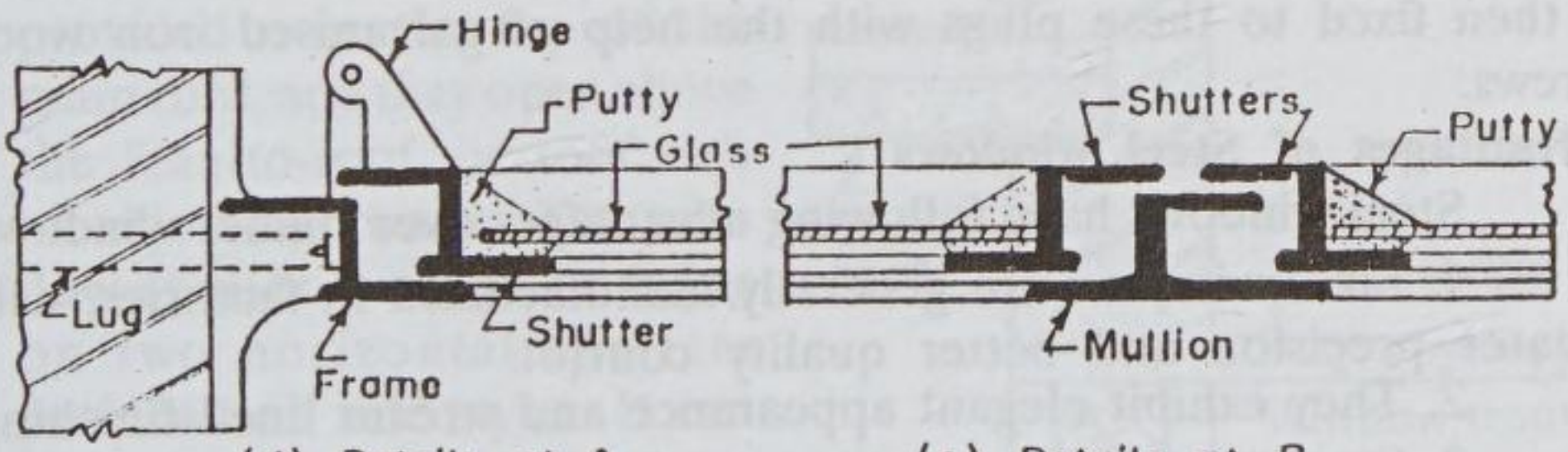
8. METAL WINDOWS

Metal windows, made of mild steel is becoming increasingly popular in private as well as public buildings, because of their strength and less cost. However, windows made of other metals, such as aluminium, bronze, stainless steel etc. are also used for those buildings where high degree of elegance finishing etc. is required. Aluminium windows are rust-proof, durable and require no maintenance and painting; they are therefore increasingly becoming popular for domestic buildings.

Fig. 17.31 shows a mild steel window. Mild steel windows are the cheapest, and are therefore extensively used in all types of buildings. Mild steel sections, used for the fabrication of metal windows, are manufactured in wide range of standard sizes. The commonly used sections are angle sections, Z-sections, T-sections and channel sections, all of which are slightly modified in shape to meet various requirements of window functioning. Steel windows can be fixed either directly to the masonry opening, or it may be fitted into wooden frame already fixed in the opening. Generally, the first alternative is adopted since it is cheaper. However, it should be ensured that no load of the wall etc., is transferred to the window frame. For this, it is usual practice to keep the size of window-opening slightly more than the window frame. Also, the frame may be fixed

in the formed opening, after the masonry work is over.





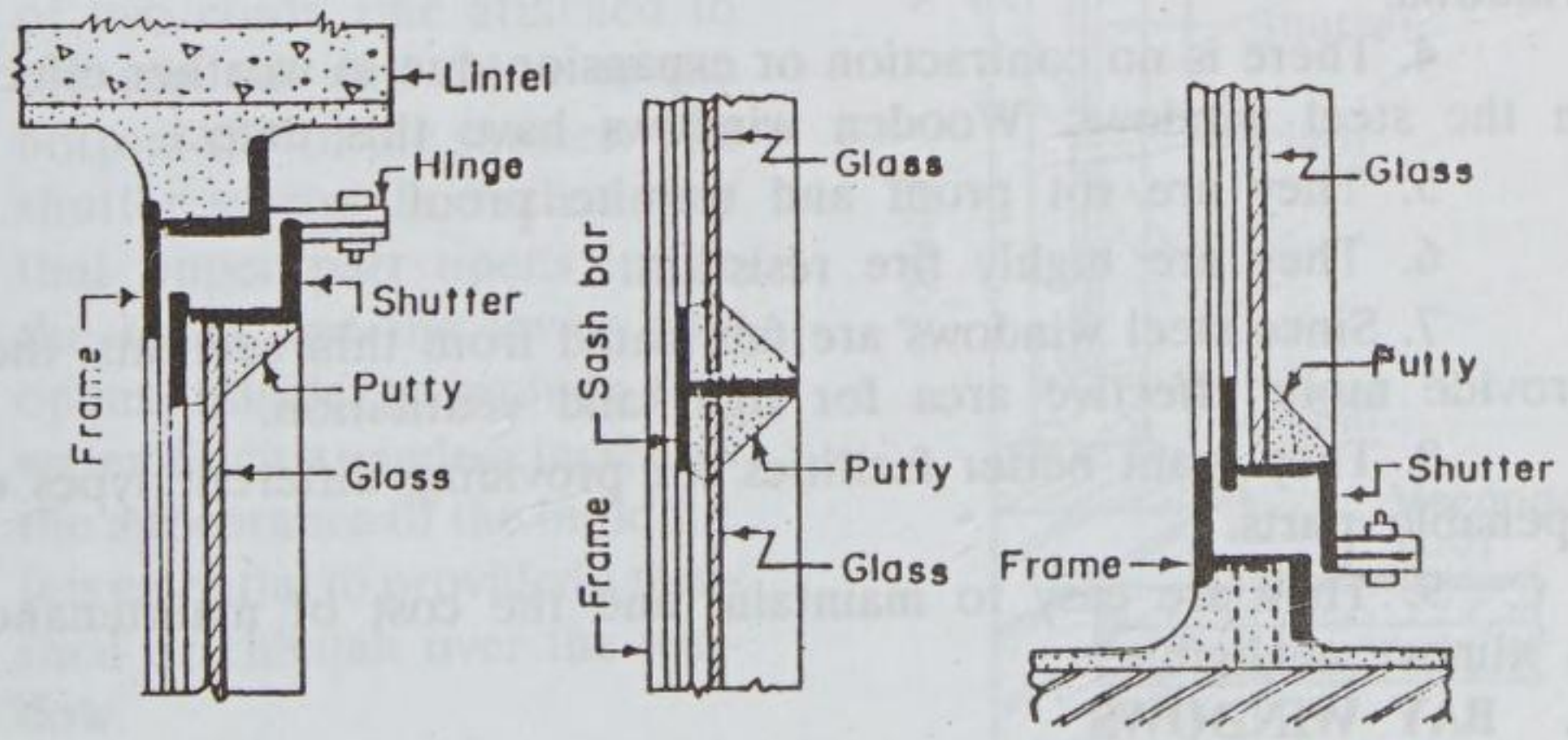


FIG. 17.31. DETAILS OF STEEL WINDOW.

Method of fixing steel windows

1. The prepared opening, in which steel window frame is to be fixed is cleared, and exact position of the window frame is marked by drawing chalk-lines along the verticals and head and sill of the window frame.

2. The distances of fixing holes are measured on the frame, and these positions are marked on the chalk lines drawn in the joints of the opening.

3. Holes are cut in the brick masonry, of size 5 cm square and 5 to 10 cm deep, to accommodate hold fasts or *lugs*.

4. The frame is placed in the opening and its position is adjusted in correct alignment by striking wooden wedges in correct position. Since there is little gap between the opening and the window frame, temporary wooden wedges can be easily driven. After adjusting the window in correct alignment, the lugs are screwed tight to the frame.

5. The lugs are grouted into the holes with cement mortar.

6. After the grout has set, wooden wedges are removed, and the space between the opening and the frame (known as *surrounds*) is filled with cement mortar.

7. In the case of stone masonry or R.C.C. work, where it is difficult to cut holes for lugs, wooden plugs are embedded at appropriate places during the construction itself. the window frame is then fixed to these plugs with the help of galvanised iron wood screws.

Advantages of Steel windows

Steel windows have following advantages over timber windows:

1. Steel windows are generally manufactured in factories, with greater precision and better quality control.

2. They exhibit elegant appearance and stream lined-finishing.

3. Steel windows are stronger and more durable than wooden windows.

4. There is no contraction or expansion due to weather effects in the steel windows. Wooden windows have this defect.

5. They are rot proof and termite proof.

6. They are highly fire resistant.

7. Since steel windows are fabricated from thin sections, they provide more effective area for light and ventilation.

8. They grant better facilities for providing different types of openable parts.

9. They are easy to maintain, and the cost of maintenance is almost negligible.

9. BAY WINDOWS

Bay windows project outside the external wall of the room.

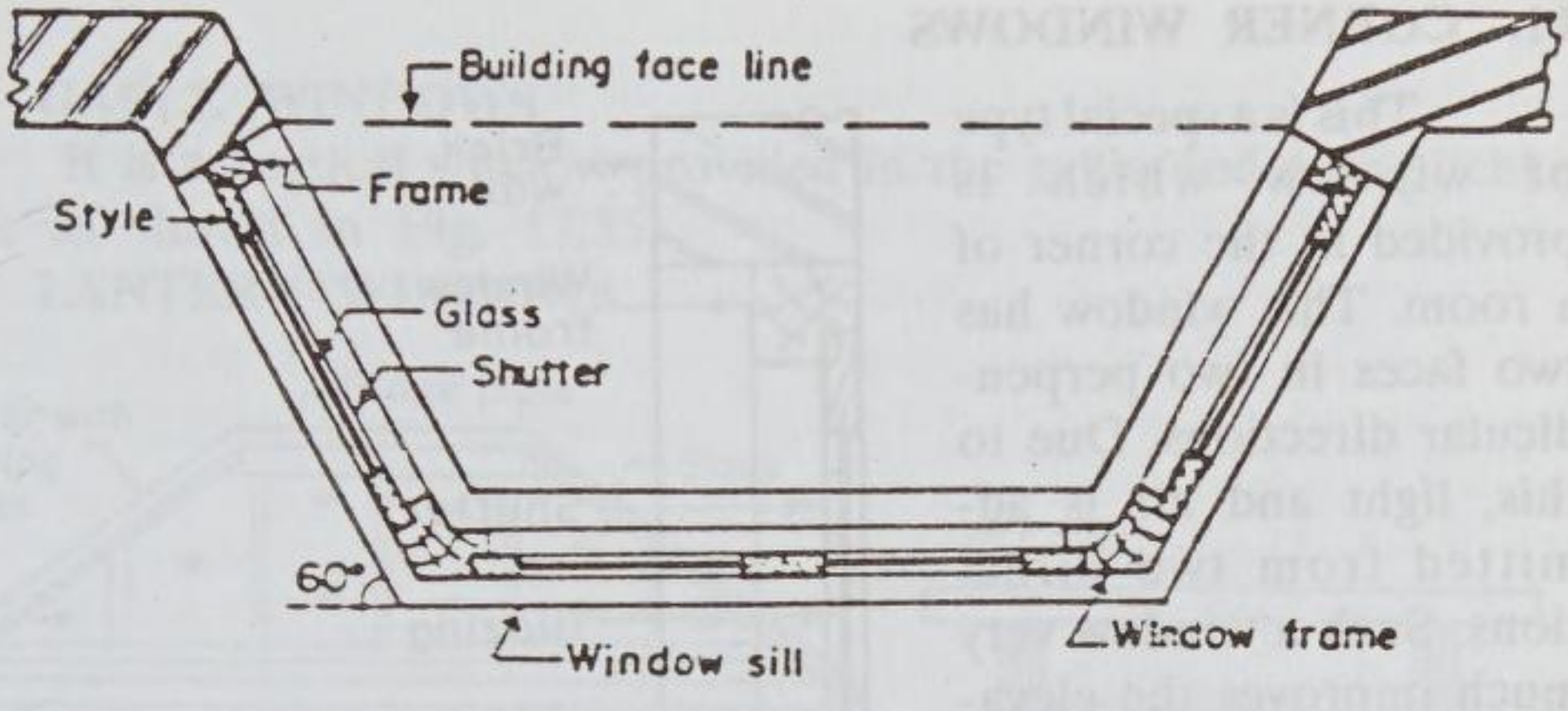


FIG. 17.32. BAY WINDOW.

This projection may be triangular, circular, rectangular or polygonal in plan. Such a window, shown in Fig. 17.32, is provided to get an increased area of opening for admitting greater light and air. They also provide extra space in the room, and improve the overall appearance of the building.

10. CLERE-STOREY WINDOW (Fig. 17.33)

These windows are provided in a room which has greater ceiling height than the surrounding rooms, or when a lean-to-roof of low height is there adjacent to the room. It is generally provided near the top of the main roof, and they open above the lean-to-roof, or roof slab of adjoining rooms. The window shutter is made to swing on two horizontal shutters provided on side styles. It can be opened or closed by means of two cords, one attached to the top rail and other to the bottom rail of the shutter. The shutter swings in such a way that upper part opens inside the room and the lower part opens outside, to exclude rain water. Such a window increases the appearance of the building. It is essential to provided a rain-shed or chhajah over the window.

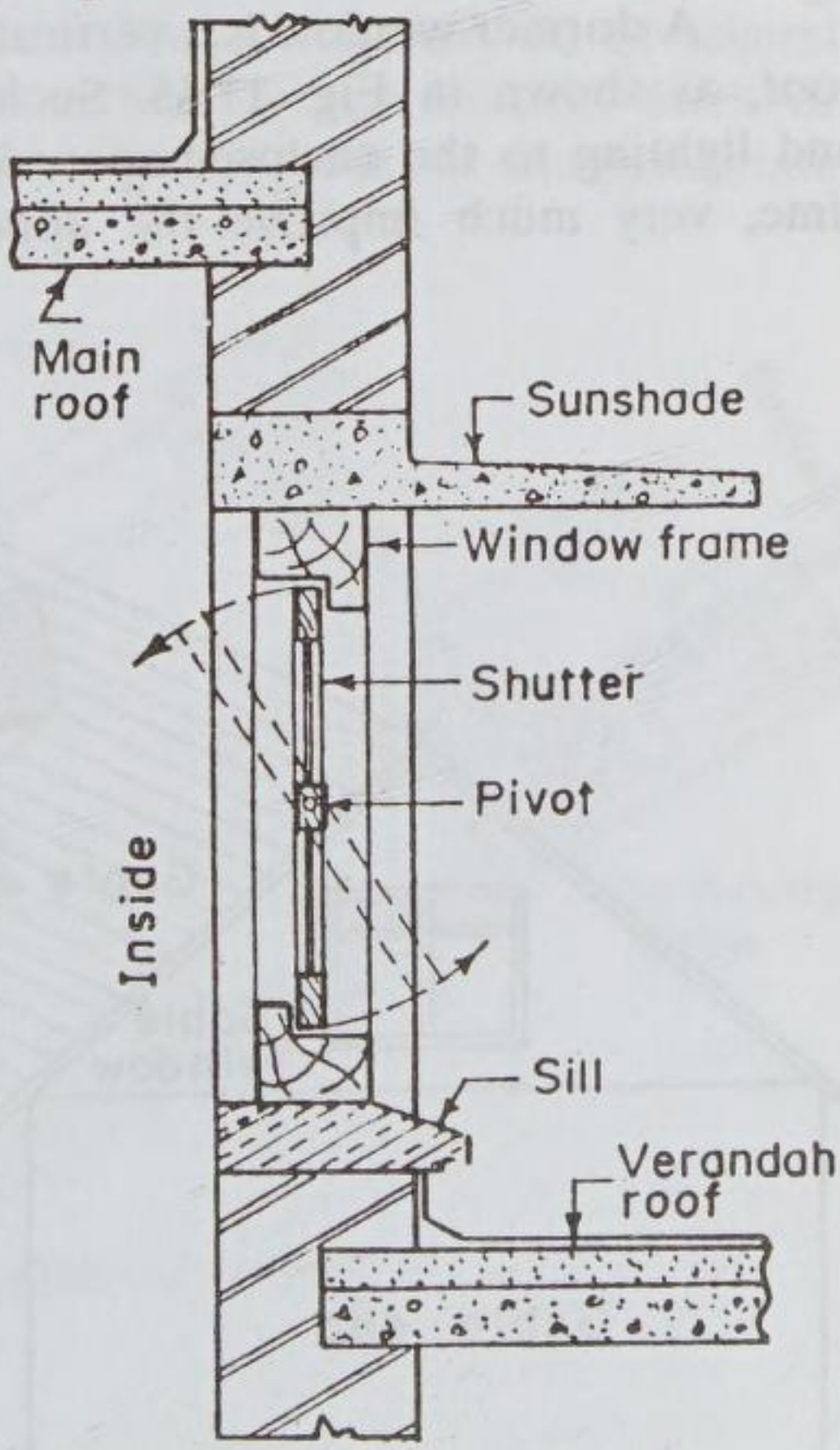


FIG. 17.33. CLERE-STOREY WINDOW.

11. CORNER WINDOWS

This is a special type of window which is provided in the corner of a room. This window has two faces in two perpendicular directions. Due to this, light and air is admitted from two directions. Such a window very much improves the elevation of the building. However, special lintel has to be cast over the window-opening. The jamb post of the window, at the corner, is made of heavy section, as shown in Fig. 17.34.

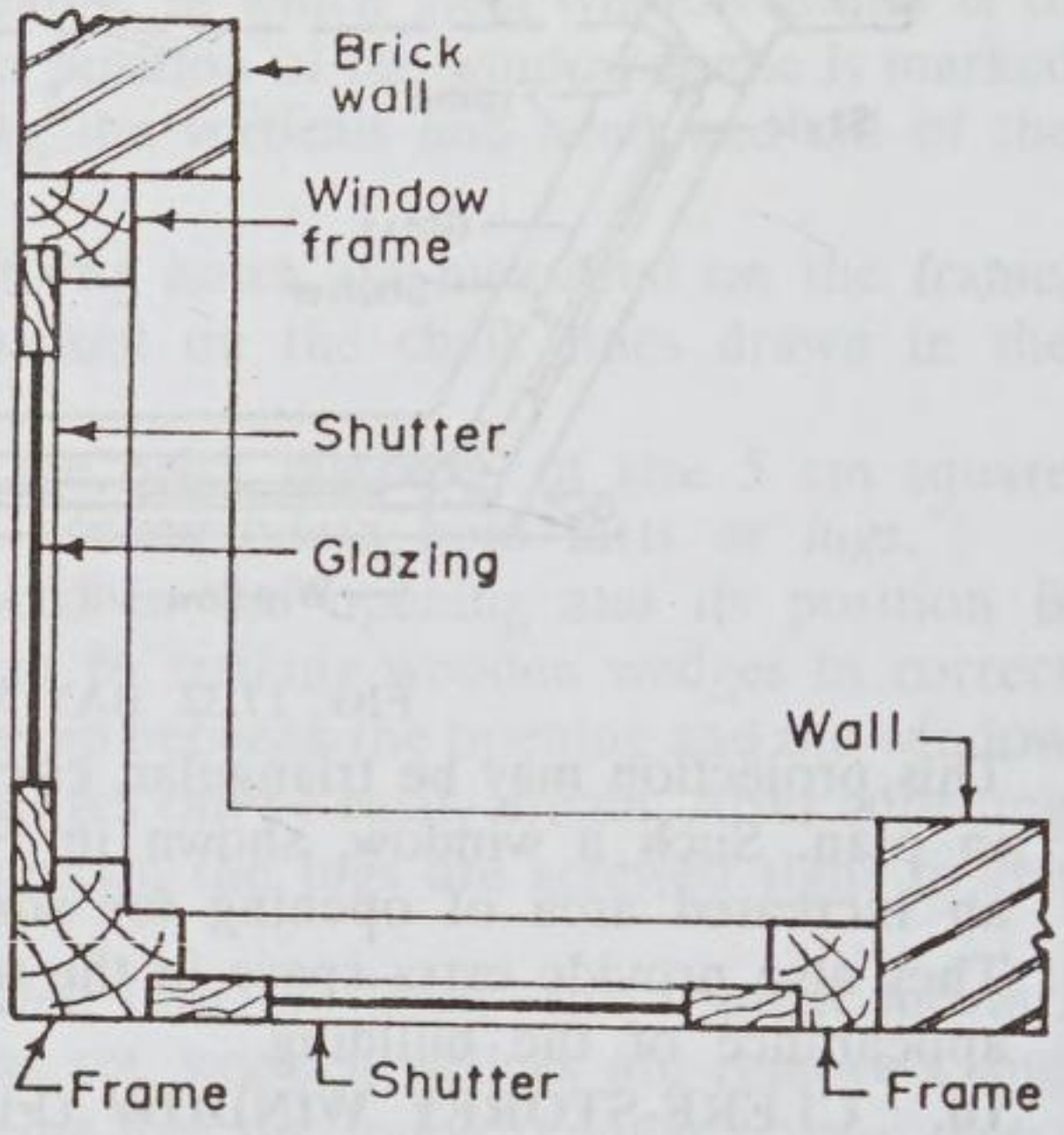


FIG. 17.34. CORNER WINDOW.

12.. DORMER WINDOWS

A dormer window is a vertical window provided on the sloping roof, as shown in Fig. 17.35. Such a window provides ventilation and lighting to the enclosed space below the roof, and at the same time, very much improve the appearance of the building.

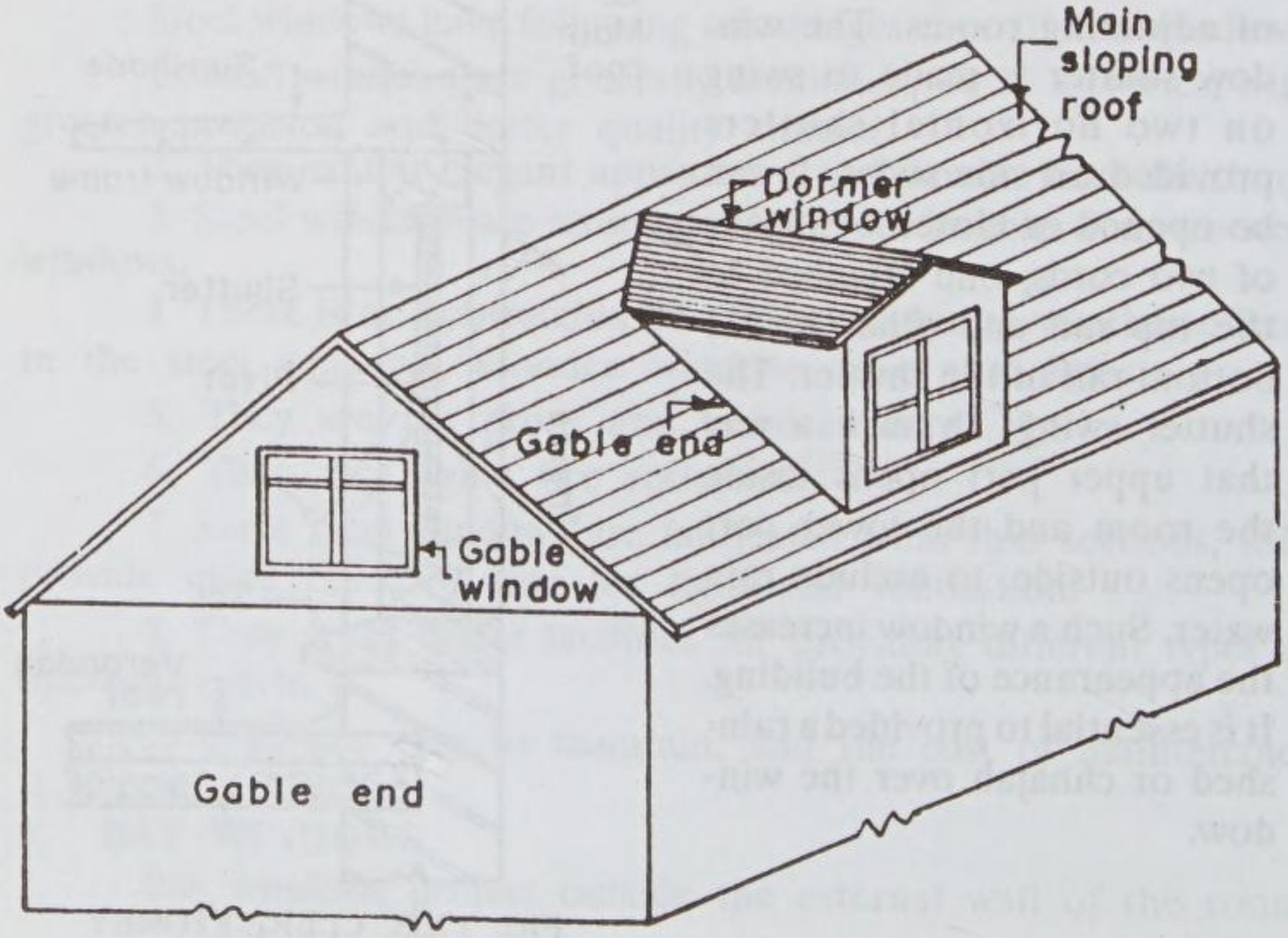


FIG. 17.35. DORMER WINDOW AND GABLE WINDOW

13. GABLE WINDOWS

It is a vertical window provided in the gable end of a pitched roof, as shown in Fig. 17.35.

14. LANTERN WINDOWS

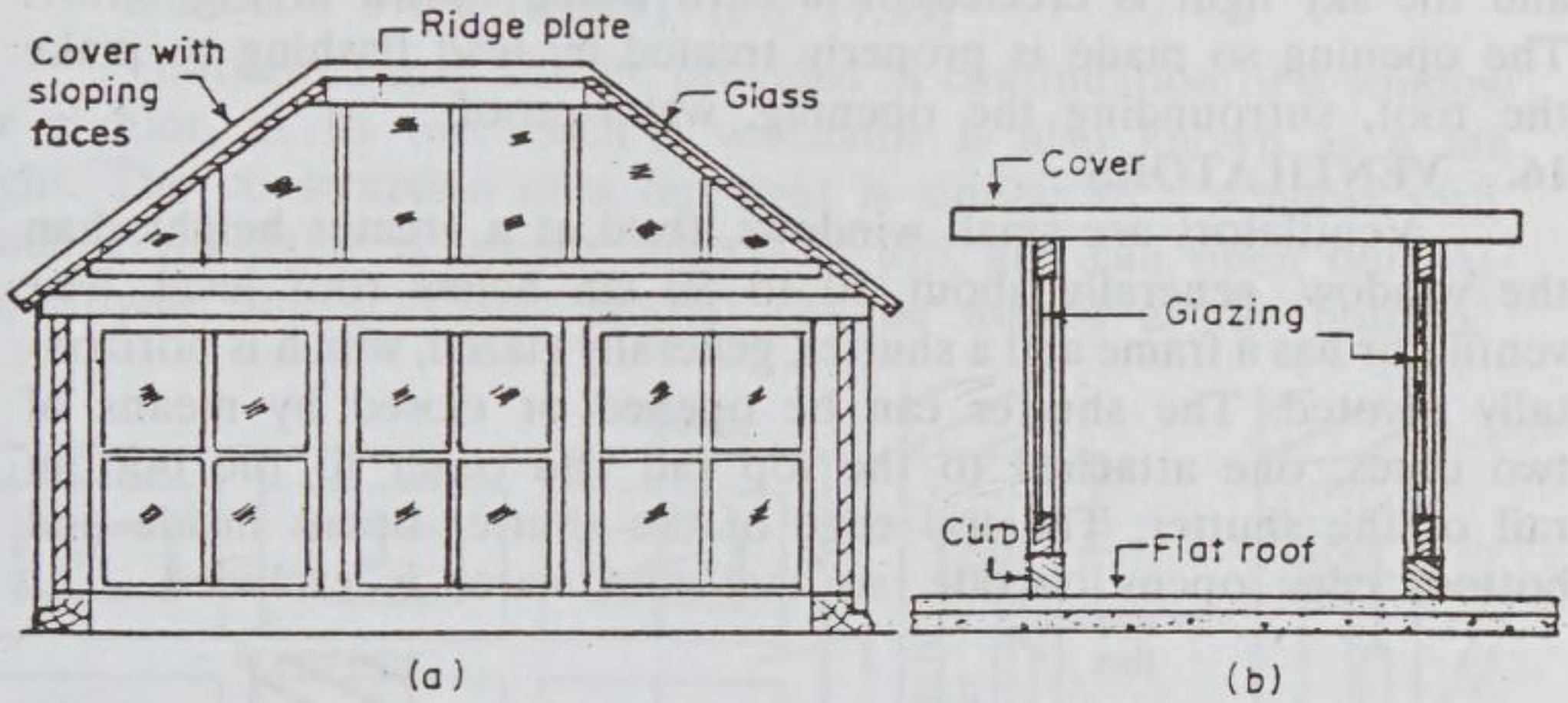
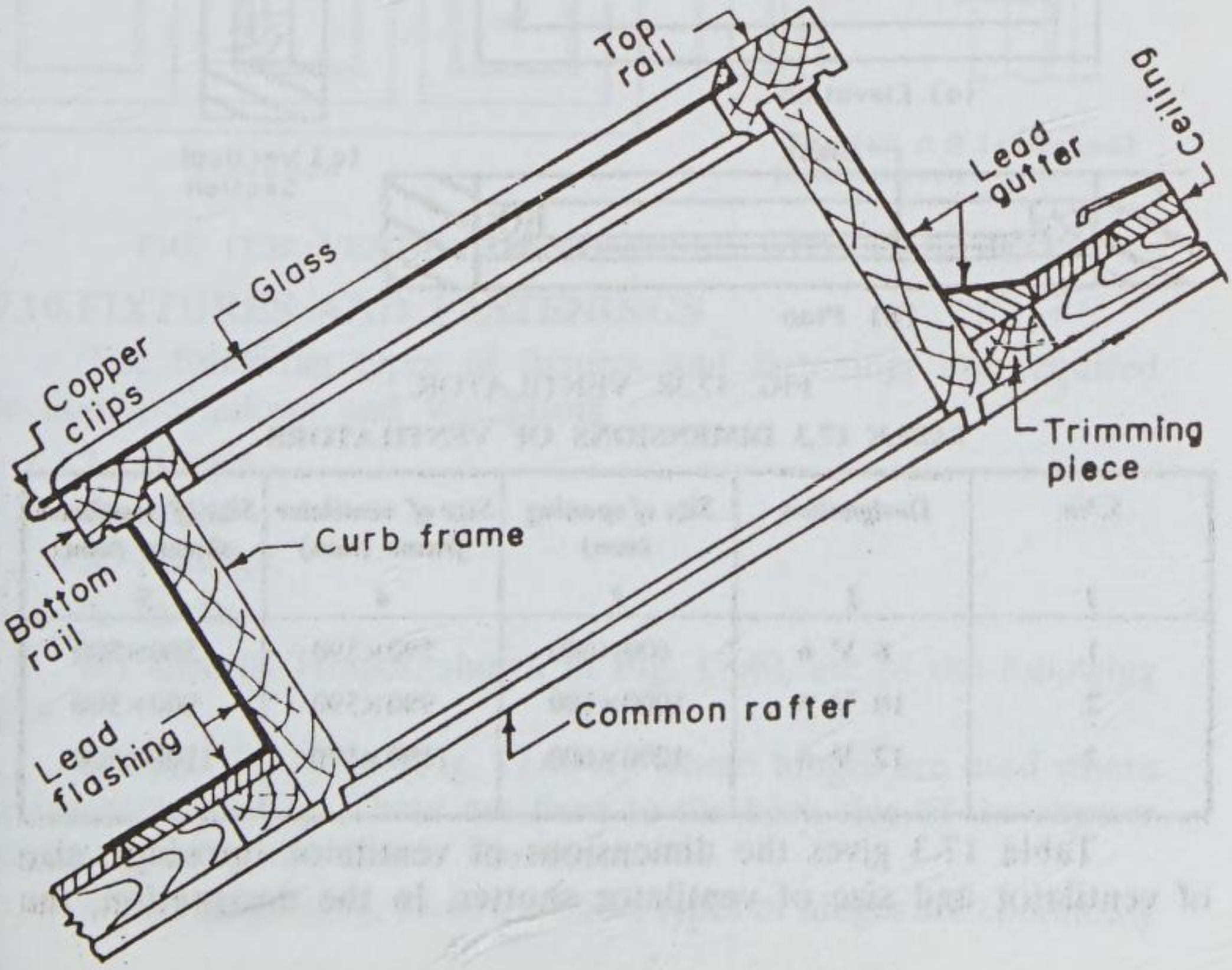


FIG. 17.36. LANTERN WINDOWS.

Such windows are provided over the flat roofs, to provide more light and air to the inner apartments/rooms of a building. The windows project above the roof level. They may be of several shapes in plan. They admit light either through vertical faces or inclined faces, as shown in Fig. 17.36 . The roof slab has an appropriate opening below the window.



15. SKY LIGHTS

A sky light is provided on a sloping roof, to admit light. The window projects above the top sloping surface. They run parallel to the sloping surface. The common rafters are suitably trimmed and the sky light is erected on a curb frame shown in Fig. 17.37. The opening so made is properly treated by lead flashing to make the roof, surrounding the opening, water-proof.

16. VENTILATORS

Ventilators are small windows, fixed at a greater height than the window, generally about 30 to 50 cm below roof level. The ventilator has a frame and a shutter, generally glazed, which is horizontally pivoted. The shutter can be opened or closed by means of two cords, one attached to the top rail and other to the bottom rail of the shutter. The top edge of the shutter opens inside and bottom edge opens outside, so that rain water is excluded.

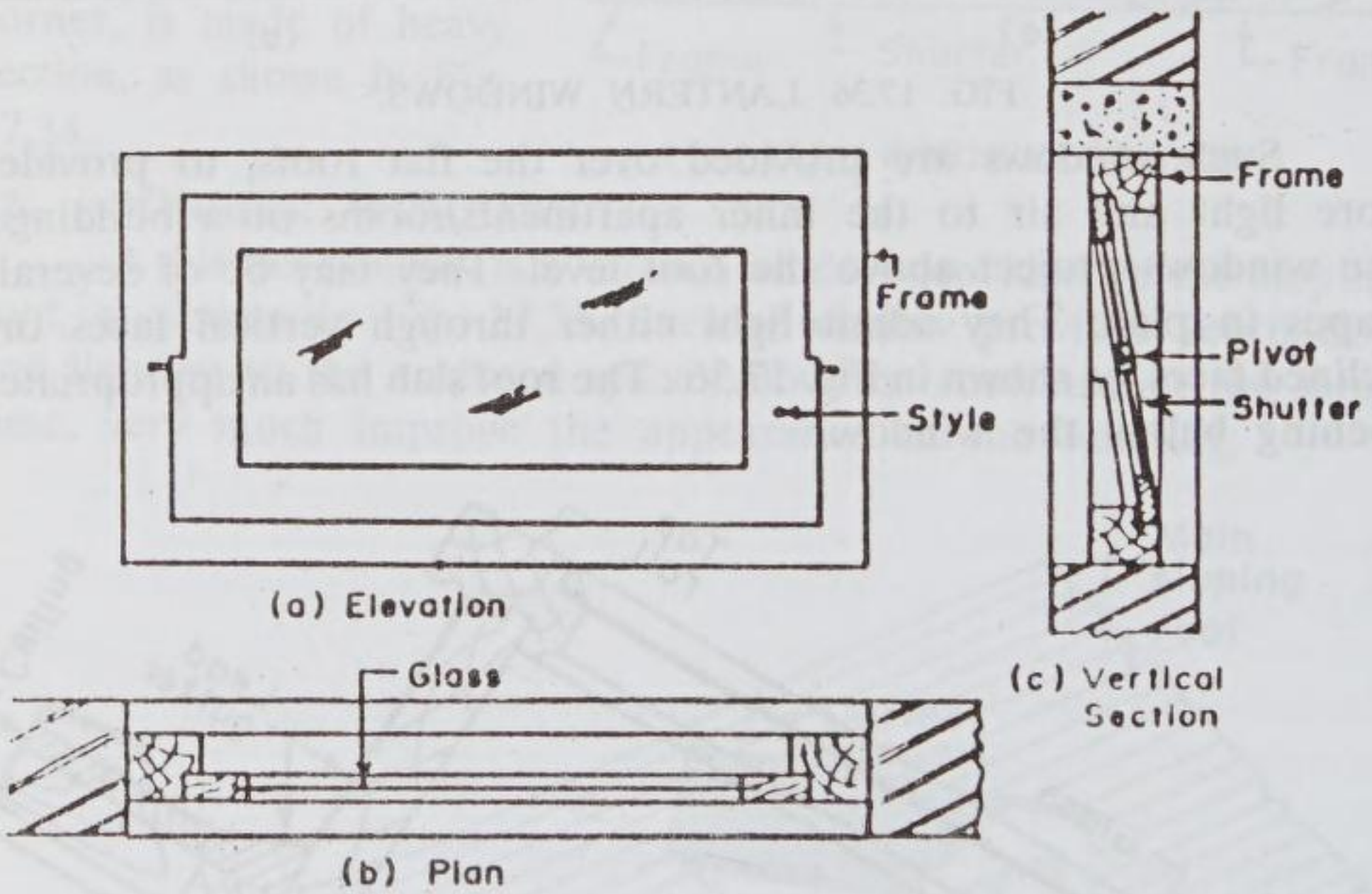


FIG. 17.38. VENTILATOR.

TABLE 17.3 DIMENSIONS OF VENTILATORS

S.No.	*Designation*	*Size of opening (mm)*	*Size of ventilator frame (mm)*	*Size of ventilator shutter (mm)*
1	*2*	*3*	*4*	*5*
1.	6 V 6	600×600	590×590	500×500
2.	10 V 6	1000×600	990×590	900×500
3.	12 V 6	1200×600	1190×590	1100×500

Table 17.3 gives the dimensions of ventilator openings, size of ventilator and size of ventilator shutter. In the designation, the

first number denotes the width of the opening in modules, each of 100 mm, letter V denotes a ventilator, while the last number denotes the height of the opening in the modules.

17.9. VENTILATOR COMBINED WITH WINDOW OR DOOR : FAN LIGHT (Fig. 17.39)

Ventilators may also be provided in continuation of a window or a door, at its top. Such a ventilator is also known as a fan light. The construction of a fan light is similar to a window sash. Such a ventilator is usually hinged at top, and can open out. Alternatively, the ventilator shutter can be hinged at the bottom.

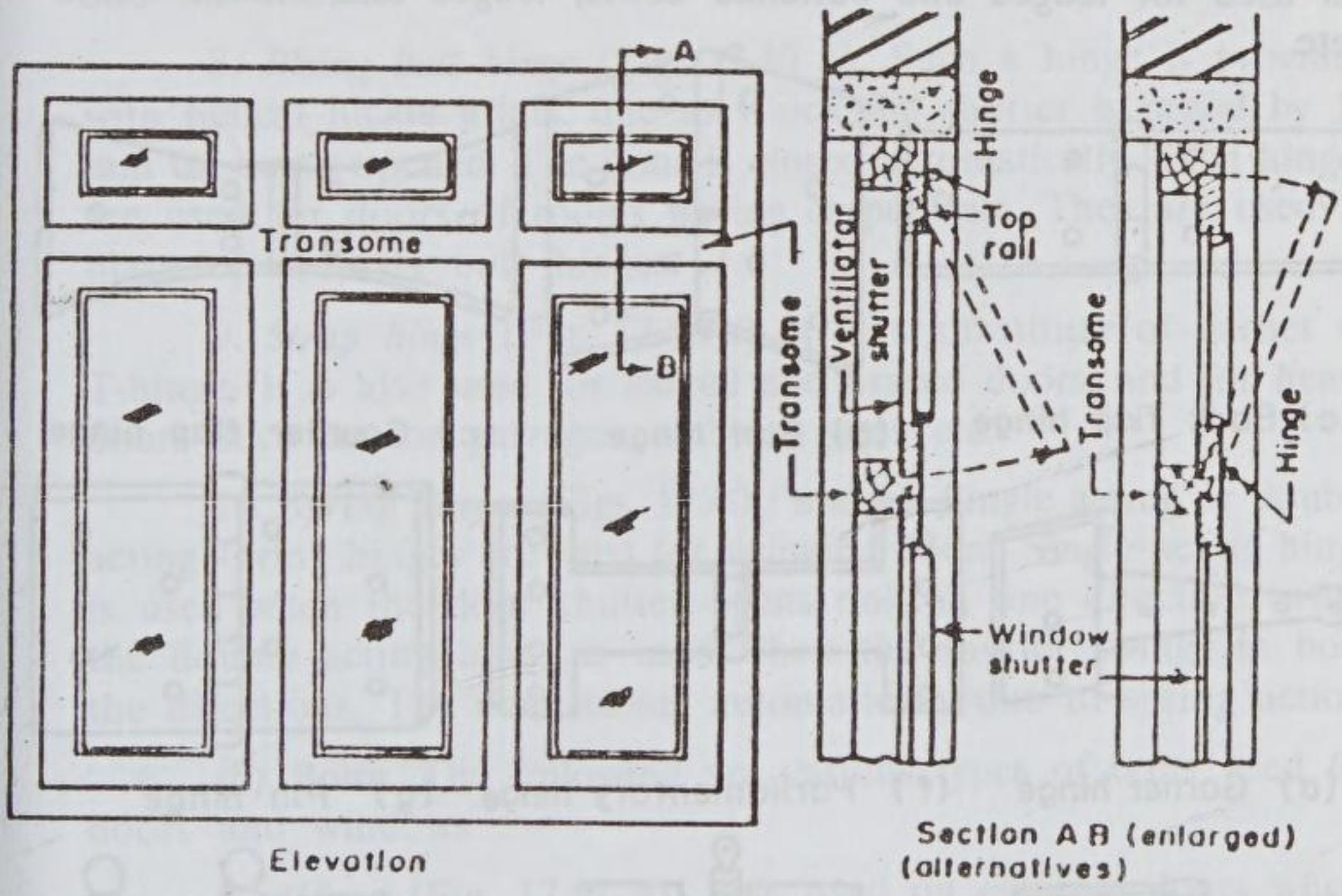


FIG. 17.39. VENTILATOR COMBINED WITH WINDOW.

17.10. FIXTURES AND FASTENINGS

The following types of fixtures and fastenings are required for doors, windows and ventilators :

1. Hinges.
2. Bolts.
3. Handles.
4. Locks.

(*a*) Hinges. Hinges, shown in Fig. 17.40, are of the following types :

1. ***Back flap hinge*** (Fig. 17.40 *a*). These hinges are used where the shutters are thin. These are fixed to the back side of the shutter and the frame, and hence the name.

2. ***Butt hinge*** (Fig. 17.40 *b*). These types of hinges are commonly

used for fixing doors and window shutters to the frame. The flanges of hinge are made of cast iron, malleable iron or steel, with counter sunk holes. One flange of hinge is screwed to the edge of the shutter while the other is screwed to the rebate of the frame.

3. ***Counter-flap hinge*** (Fig. 17.40 *c*). This hinge is formed in three parts and has two centres. Hence the two leaves can be folded back to back.

4. ***Garnet hinge*** (Fig. 17.40 *d*). This type of hinge, also known as T-shutter, has a long arm which is screwed to the shutter, and a short arm or plate which is screwed to the door frame. The hinge is used for ledged and battened doors, ledged and braced doors etc.

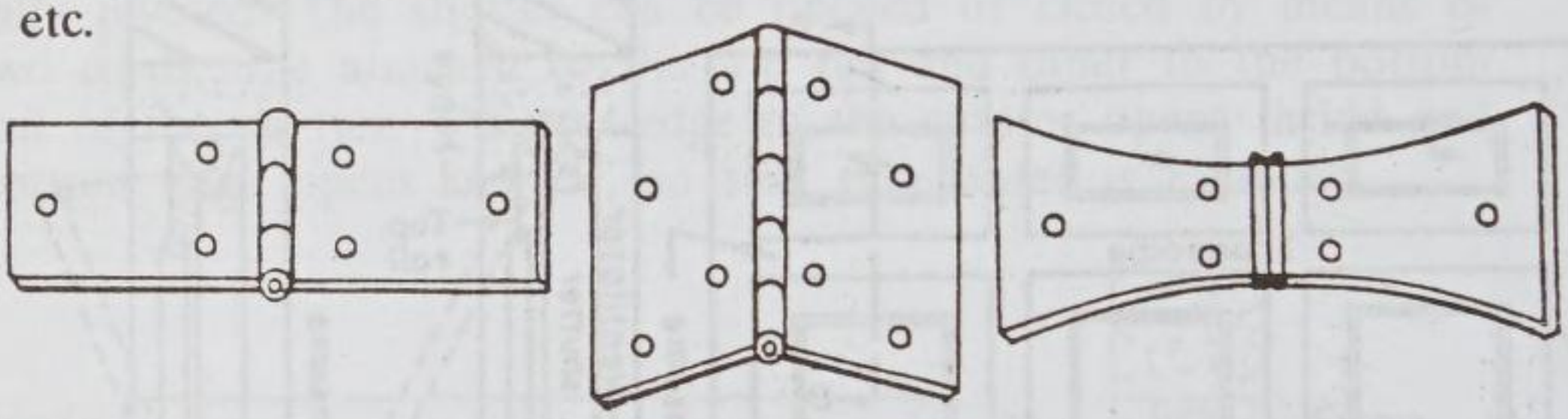


FIG. 17.40. HINGES.

5. ***Nar-madi hinge*** (Fig. 17.40 *e*) This is used for heavy doors. The flange or strap of the hinge is fixed to the door shutter while the pin on which the strap rotates is fixed to the frame.

6. ***Parliamentary hinge*** (Fig. 17.40 *f*). These hinges permit the door shutters, when open, to rest parallel to the wall. Hence these hinges are used when the opening is narrow and when it is required to keep the opening free from obstruction due to door shutters.

7. ***Pin hinge*** (Fig. 17.40 *g*). This is also used for heavy door-shutters. The centre pin of the hinge can be removed and the two leaves or straps of the hinge can be fixed separately to the frame and the shutter.

8. ***Rising butt hinge*** (Fig. 17.40 *h*). Such a hinge is provided with helical nickle joints, due to which the shutter is raised by 10 mm on being opened. The door is closed automatically. Such hinges are used for doors of rooms having carpets etc. They are used in place of ordinary butt hinges.

9. ***Strap hinge*** (Fig. 17.40 *i*) It is a substitute of garnet or T-hinge. It is also used for ledged and braced doors, and for heavy doors such as for garrages, stables, gates etc.

10. ***Spring hinges*** (Figs. 17.40 *j* and *k*). Single acting or double acting spring hinges are used for swinging doors. Single acting hinge is used when the door shutter opens only in one direction, while the double acting hinge is used when the shutter swings in both the directions. The door closes automatically due to spring action.

(*b*) **Bolts**. The following are various types of bolts used for doors and windows :

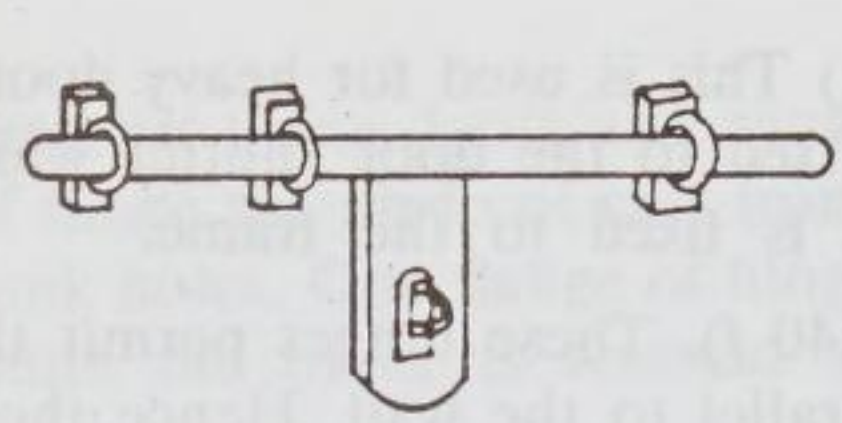
1. ***Aldrop*** (Fig. 17.41 *a*). It is fixed on external doors where pad locks are to be used.

2. ***Barrel bolt*** (Fig. 17.41 *b*). It is used for fixing back faces of doors. The socket is fixed to the door frame while the plate is screwed to the inside of the shutter.

3. ***Espagnolette bolt*** (Fig. 17.41 *c*). This is used for securing high doors and casement windows, the top of which cannot be easily reached.

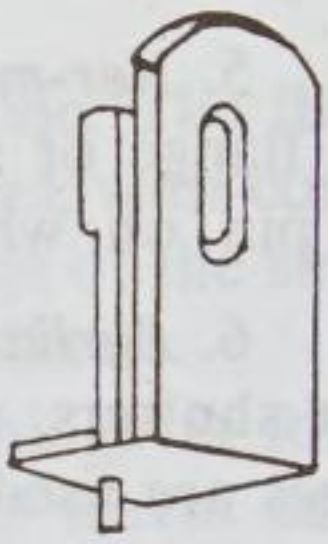
4. ***Flush bolt*** (Fig. 17.41 d). This bolt is used when it is desired to keep the bolt flush with the face of the door.

5. ***Hasp and staple bolt*** (Fig. 17.41 *e*) This is also used for external doors where pad lock is to be used. The staple is fixed to the door frame, while the hasp fixed to the shutter.

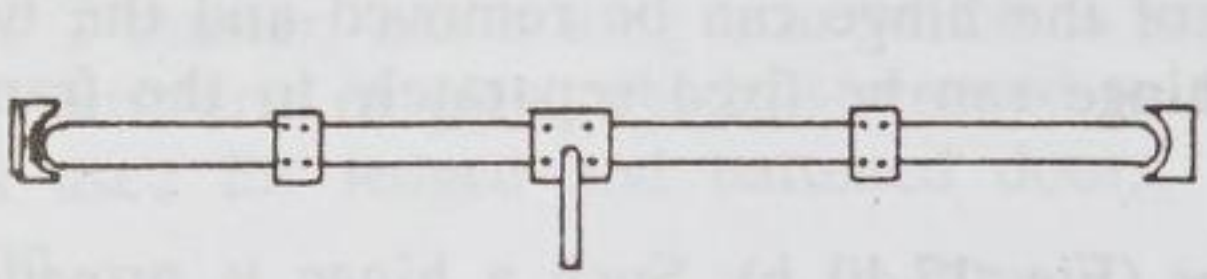


(a) Aldrop bolt

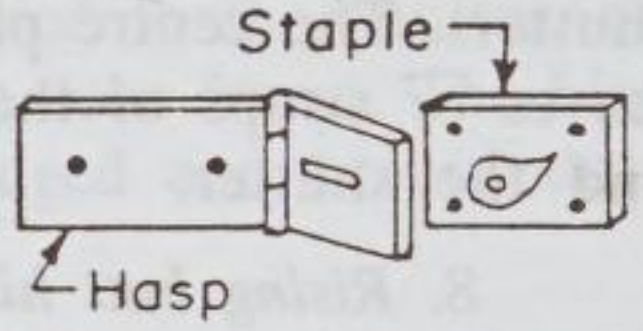
(b) Barrel bolt



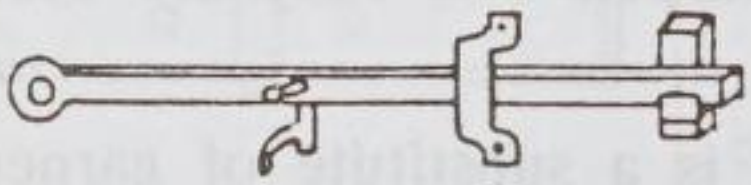
(d) Flush bolt



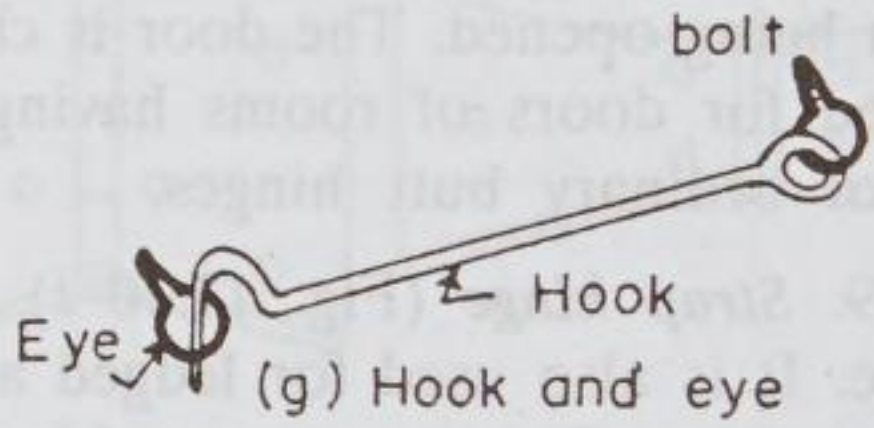
(c) Espagnalette bolt



(e) Hasp and staple bolt



(f) Latch



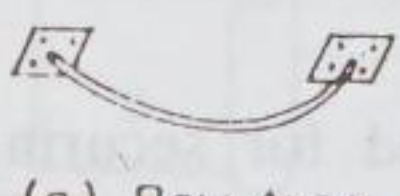
(g) Hook and eye

FIG. 17.41. BOLTS.

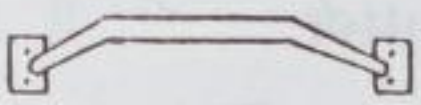
6. ***Latch*** (Fig. 17.41 *f*). This is made of malleable iron or bronze. It consists of lever pivoted at one end, which can be actuated by a trigger passing through the shutter ; the lever is secured in a hasp and staple. It is fixed to the inside face of the door.

7. ***Hook and eye*** (Fig. 17.41 *g*). This is used for keeping the window shutter in position when the window is open. The hook is fixed to the sill of the frame while the eye is fixed to the bottom rail of the shutter.

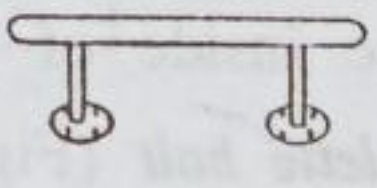
(*c*) **Handles.** Various types of handles are shown in Fig. 17.42.



(a) Bow type



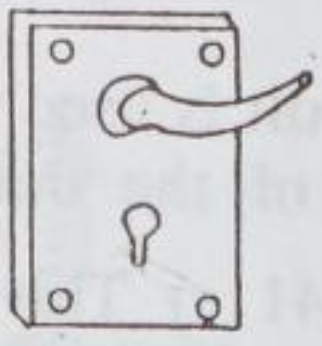
(b) Bow type



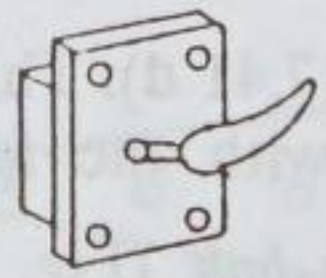
(c)

(d)

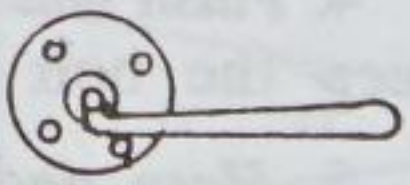
(e) Ward-robe handle



(f) Lever handle



(g) Door handle



(h) Lever hahdle

FIG. 17.42. DOOR HANDLES.

(d) **Locks.** Commonly used locks are illustrated in Fig. 17.43.

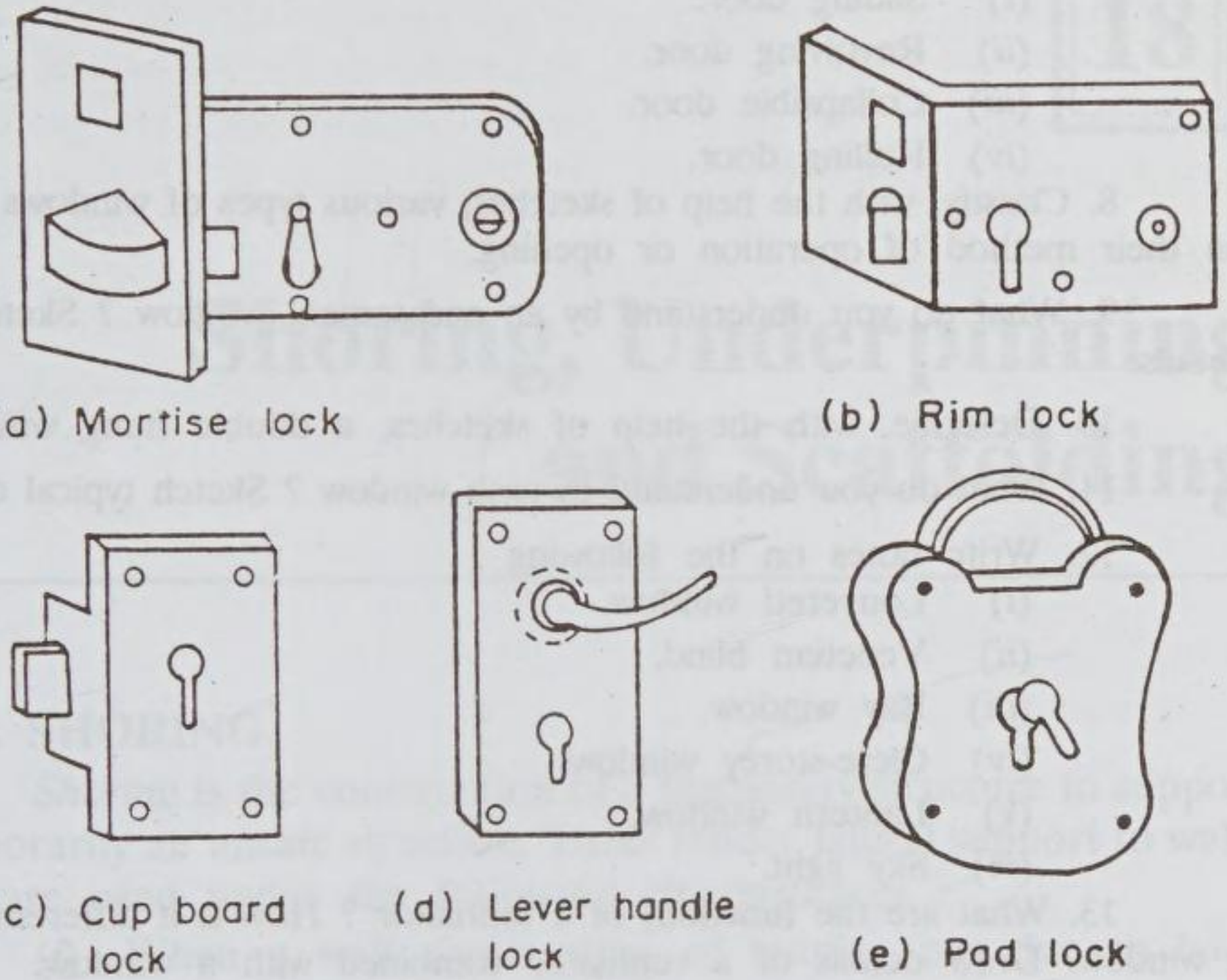


FIG. 17.43. LOCKS.

PROBLEMS

1. Write a note on location of doors and windows.
2. Define the following terms :
   (*i*) Mullion.
   (*ii*) Transom.
   (*iii*) Reveal.
   (*iv*) Rebate.
   (*v*) Style.
   (*vi*) Horn.
   (*vii*) Sash bar.
3. How the sizes of doors and windows are fixed ?
4. Draw a neat sketch of a timber door with shutter, label various parts, and draw the details of various joints.
5. Explain, with sketches, the following types of doors :
   (*i*) Battened, ledged, braced and framed doors.
   (*ii*) Framed and panelled doors.
   (*iii*) Flush doors.
   (*iv*) Louvered doors.
6. Compare wooden and steel doors. Draw the detail of each type.

7. Write notes on the following :
   (*i*) Sliding door.
   (*ii*) Revolving door.
   (*iii*) Collapsible door.
   (*iv*) Rolling door.

8. Classify, with the help of sketches, various types of windows based on their method of operation or opening.

9. What do you understand by an encasement window ? Sketch the details.

10. Describe, with the help of sketches, a double hung window.

11. What do you understand by sash window ? Sketch typical details.

12. Write notes on the following :
   (*i*) Louvered window.
   (*ii*) Venetian blind.
   (*iii*) Bay window.
   (*iv*) Clere-storey window.
   (*v*) Lantern window.
   (*vi*) Sky light.

13. What are the functions of a ventilator ? How is it different from a window. Draw details of a ventilator combined with a window.

14. Write a note on various types of hinges used for doors and windows.

15. Write a note on various types of bolts used for doors and windows.

16. Differentiate between the following :
   (*i*) Fixed and pivoted windows.
   (*ii*) Sky light and Dormer window.
   (*iii*) Ventilator and clere-storey window.
   (*iv*) Corner window and bay window.

17. (*a*) Compare steel and timber windows.
   (*b*) Draw details of a steel window.

18

Shoring, Underpinning and Scaffolding

18.1. SHORING

Shoring is the construction of a temporary structure to support temporarily an unsafe structure. These render lateral support to walls and are used under the following circumstances :

(*i*) When a wall shows signs of bulging out due to bad workmanship.
(*ii*) When a wall cracks due to unequal settlement of foundation, and the cracked wall needs repairs.
(*iii*) When an adjacent structure is to be dismantled.
(*iv*) When openings are to be made or enlarged in the wall.

Shores may be of the following types :

1. Raking Shores.
2. Flying Shores.
3. Dead Shores.

18.2. RAKING SHORES

In this method, inclined members, called *rakers* are used to give lateral support to the wall, as shown in Fig. 18.1. A raking shore consists of the following components : (*i*) rakers or inclined members, (*ii*) wall plate, (*iii*) needles, (*iv*) cleats (*v*) bracing and (*vi*) sole plate. The wall plate, about 20 to 25 cm wide and 5 to 7.5 cm thick is placed vertically along the face of the wall and is secured by means of needles of 10 cm × 7.5 cm section. These needles penetrate the wall by about 10 cm. In order that the needles do not get sheared off due to the thrust of the raker, the needles are further strengthened by means of cleats which are nailed directly to the wall plate. Rakers abut against the needles in such a way that the centre line of the raker and the wall meet at the floor level. Thus, there will be one raker corresponding to each floor.

These rakers are inter- connected by struts, to prevent their buckling. The feet of rakers are connected to an inclined *sole plate*, embedded into the ground by means of *iron dogs*. The feet of rakers are further stiffened near the sole plate by means of hoop iron. The wall plate distribute the pressure to the wall uniformly.

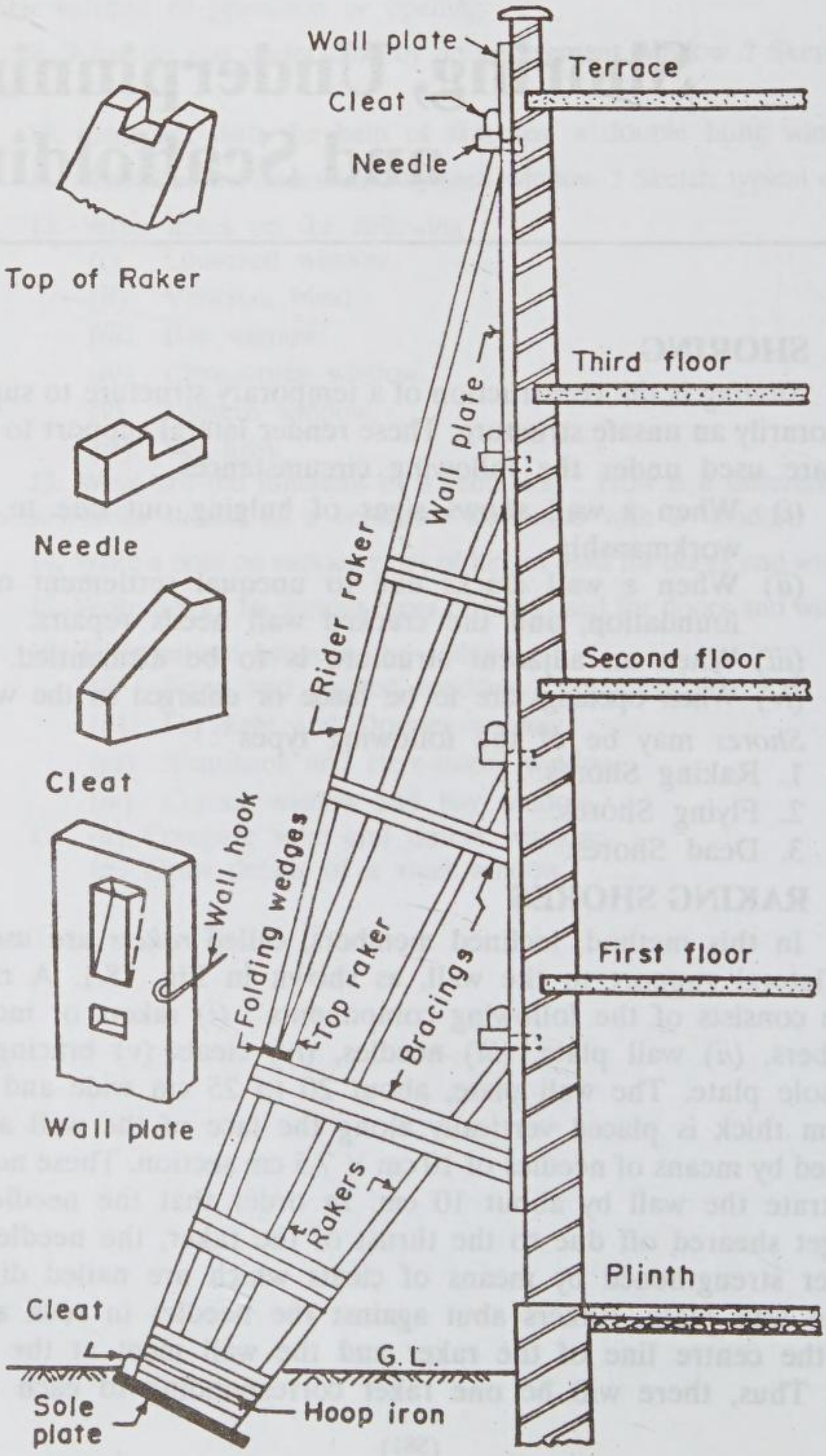


FIG. 18.1. RAKING SHORE.

The following *points* are note worthy :

1. Rakers should be inclined to the ground by 45°, to make them more effective. However, in practice, the angle may vary from 45° to 75°. The top raker should not be inclined steeper than 75°.

2. For tall buildings, the length of raker can be reduced by introducing *rider raker.*

3. Rakers should be properly braced at intervals.

4. The size of the rakers should be decided on the basis of anticipated thrust from the wall.

5. The centre line of a raker and the wall should meet at floor level.

6. If longer length of the wall needs support, shoring may be spaced at 3 to 4.5 m spacing, depending upon the requirements.

7. The sole plate should be properly embedded into the ground, at an inclination, and should be of proper section. The size of the sole plate should be such that it accommodates all the rakers, and a cleat provided along the outer edge.

8. Wedges should not be used on sole plates since they are likely to give way under vibrations which are likely to occur.

18.3 . FLYING OR HORIZONTAL SHORES

Such shores are used to give horizontal support to two adjacent, parallel party walls which have become unsafe due to removal or collapse of the intermediate building. All types of arrangements of supporting the unsafe structure in which the shores do not reach

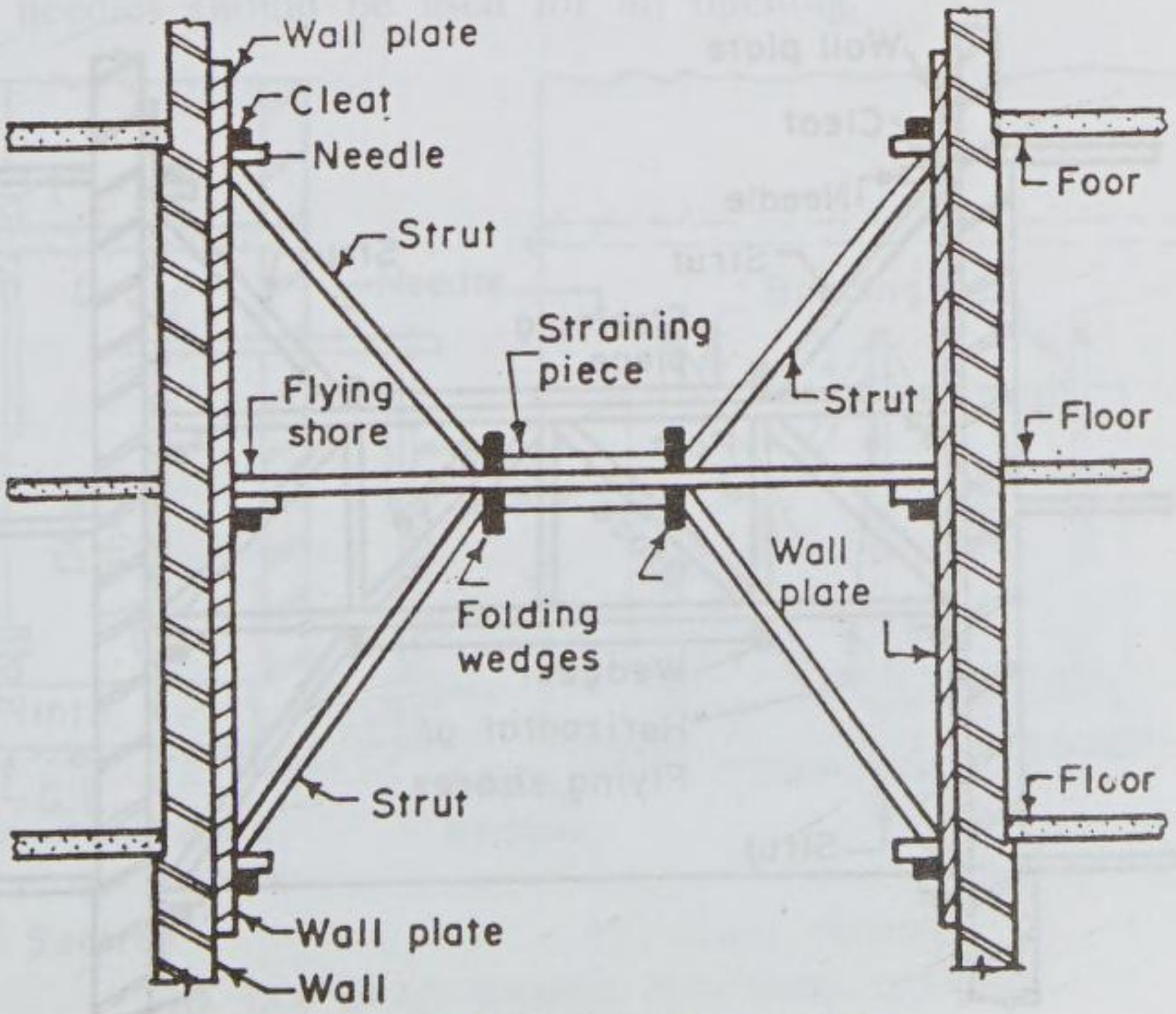


FIG. 18.2 SINGLE FLYING SHORE.

the ground fall under this category. If the walls are quite near to each other (distance upto 9 m), *single flying shore* (Fig. 18.2) can be constructed. It consists of wall plates, needles, cleats, struts, horizontal shore, straining pieces and folding wedges. When the distance between the walls is more, a *compound* or *double flying shore* (Fig. 18.3) may be provided. Flying shores have the advantage that building operations of the ground are not obstructed.

The following points should be kept in mind while erecting the flying shores :

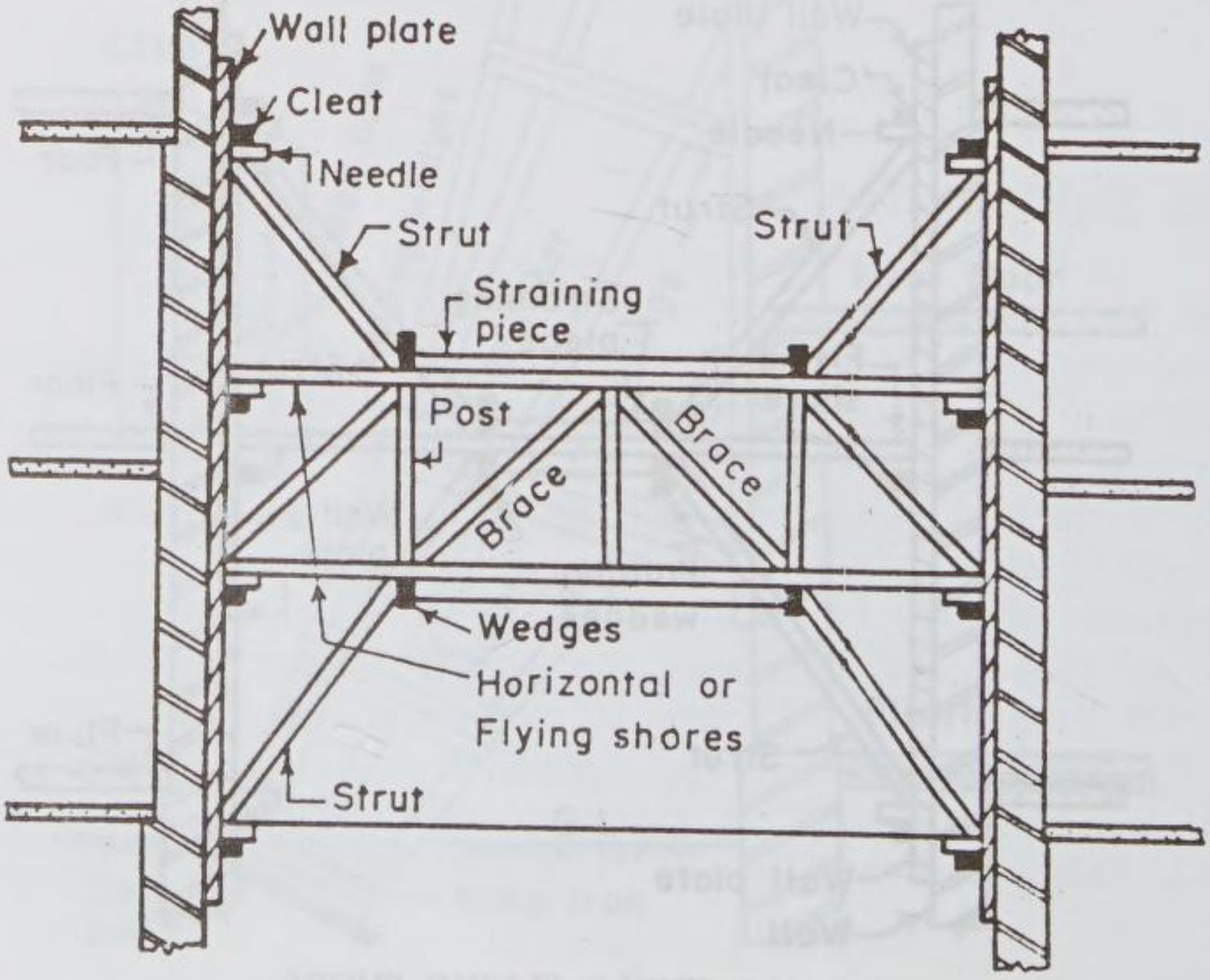
1. The centre lines of flying shore and struts and those of the walls should meet at floor levels of the two buildings. If the floor levels are different, the horizontal shore should be placed either mid-way between the levels of the two floor of equal strength, or it should be placed at the level of weaker floor.

2. The struts should preferably be inclined at 45°. In no case should this inclination exceed 60°.

3. Single shores should be used only upto 9 m distance between walls. For greater distance, double shores should be provided. In that case, both the horizontal shores should be symmetrically placed with respect to the floor levels.

4. The flying shores should be spaced at 3 to 4.5 m centres, along the two walls, and horizontal braces should be introduced between adjacent shores.

5. Large factor of safety should be used for determining sections of various members of the shoring, since it is very difficult to assess



18.3 DOUBLE OR COMPOUND FLYING SHORE.

the actual loads.

6. Flying shores are inserted when the old building is being removed, and should be kept in position till the new unit is constructed.

18.4. DEAD OR VERTICAL SHORES

Such type of shoring consists of vertical members known as *dead shores* supporting horizontal members known as *needles*.

The needles transfer the load of the wall etc. to the dead shores. Such shoring is provided to serve the following purposes :

1. To rebuild the defective lower part of the wall.

2. To rebuild or deepen the existing foundation.

3. To make large opening in the existing wall at lower level.

Holes are made in the wall at suitable height. Needles, which are made of thick wooden sections or of steel, are inserted in the holes. Each needle is supported at its two ends by vertical posts or dead shores. The dead shores stand away from the walls so that repair work is not obstructed. The shores are supported on sole plates and folding wedges.

The following points are note worthy :

1. The section of needles and dead shores should be adequate to transfer the load, which can estimated with fair degree of accuracy.

2. The needles are spaced at 1 to 2 metres. A minimum of three needles should be used for an opening.

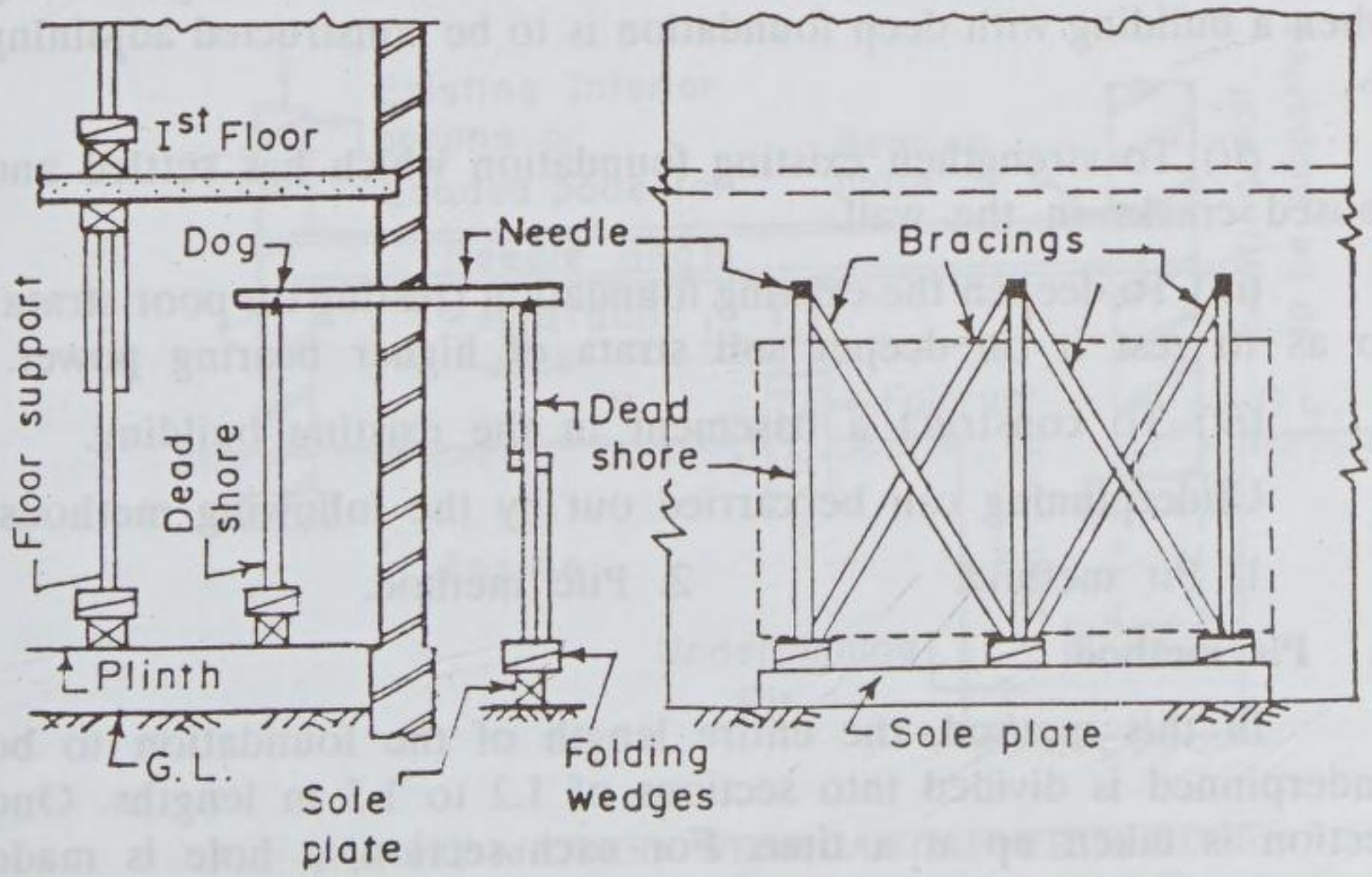


FIG. 18.4. DEAD SHORES (VERTICAL SHORES).

3. The needles should be suitably braced.

4. If the opening is made in an external wall, the length of outer dead shores will be greater than the inner ones.

5. The dead shores are supported on sole plates. Folding wedges should be inserted between the two. It is preferable to use one single sole plate between dead shores in a row.

6. The floors should be suitably supported from inside.

7. If the external wall is weak, raking shores may be provided, in addition to the dead shores.

8. Shores should be removed only when the new work has gained sufficient strength, but in no case earlier than 7 days of the completion of new work. The new work should have proper strutting.

9. The sequence of removal should be (*i*) needles (*ii*) strutting from opening (*iii*) floor-strutting inside and (*iv*) raking shore if any. An interval of 2 days should be allowed between each one of these removal operations.

18.5. UNDERPINNING

The process of placing a new foundation under an existing one or strengthening an existing foundation is called *underpinning* of foundations.

Underpinning may be required to serve the following purposes :

(*i*) To strengthen the shallow foundation of existing building when a building with deep foundation is to be constructed adjoining it.

(*ii*) To strengthen existing foundation which has settled and caused cracks in the wall.

(*iii*) To deepen the existing foundation (resting on poor strata) so as to rest it on deeper soil strata of higher bearing power.

(*iv*) To construct a basement in the existing building.

Underpinning can be carried out by the following methods:

1. Pit method. 2. Pile method.

1. Pit method

In this method, the entire length of the foundation to be underpinned is divided into sections of 1.2 to 1.5 m lengths. One section is taken up at a time. For each section, a hole is made in the wall, above the plinth level, and needle is inserted in the hole. Needles may be either of stout timber or steel section. Bearing

plates are placed above the needle to support the masonry above it. Needle is supported on either side of the wall on crib supports (wooden blocks) and screw jacks. The foundation pit is then excavated upto the desired level and new foundation is laid. When the work of one section is over, work on next to next section is taken up *i.e.*. alternate sections are underpinned in the first round, and then the remaining sections are taken up. Fig. 18.5 shows the section. If the wall to be underpinned is weak, raking shores may be provided. Similarly, the floors may also be supposed, if required.

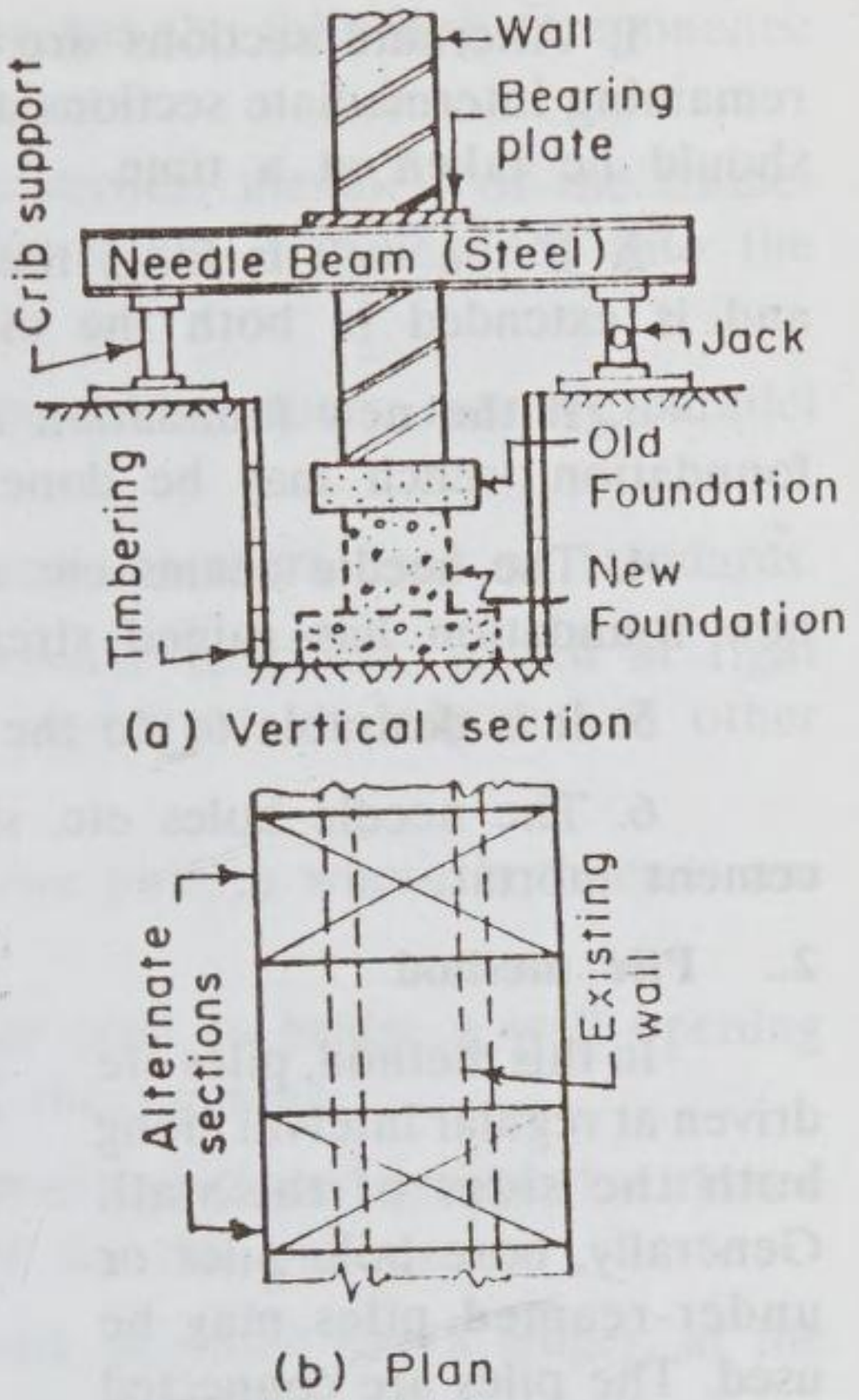


FIG. 18.5. PIT METHOD.

If an interior strong column exists, or if the foundation is to be extended only to one side, cantilever needle beams may be used in the place of central needle beam, as shown in Fig. 18.6. Jack is placed between the column and the wall.

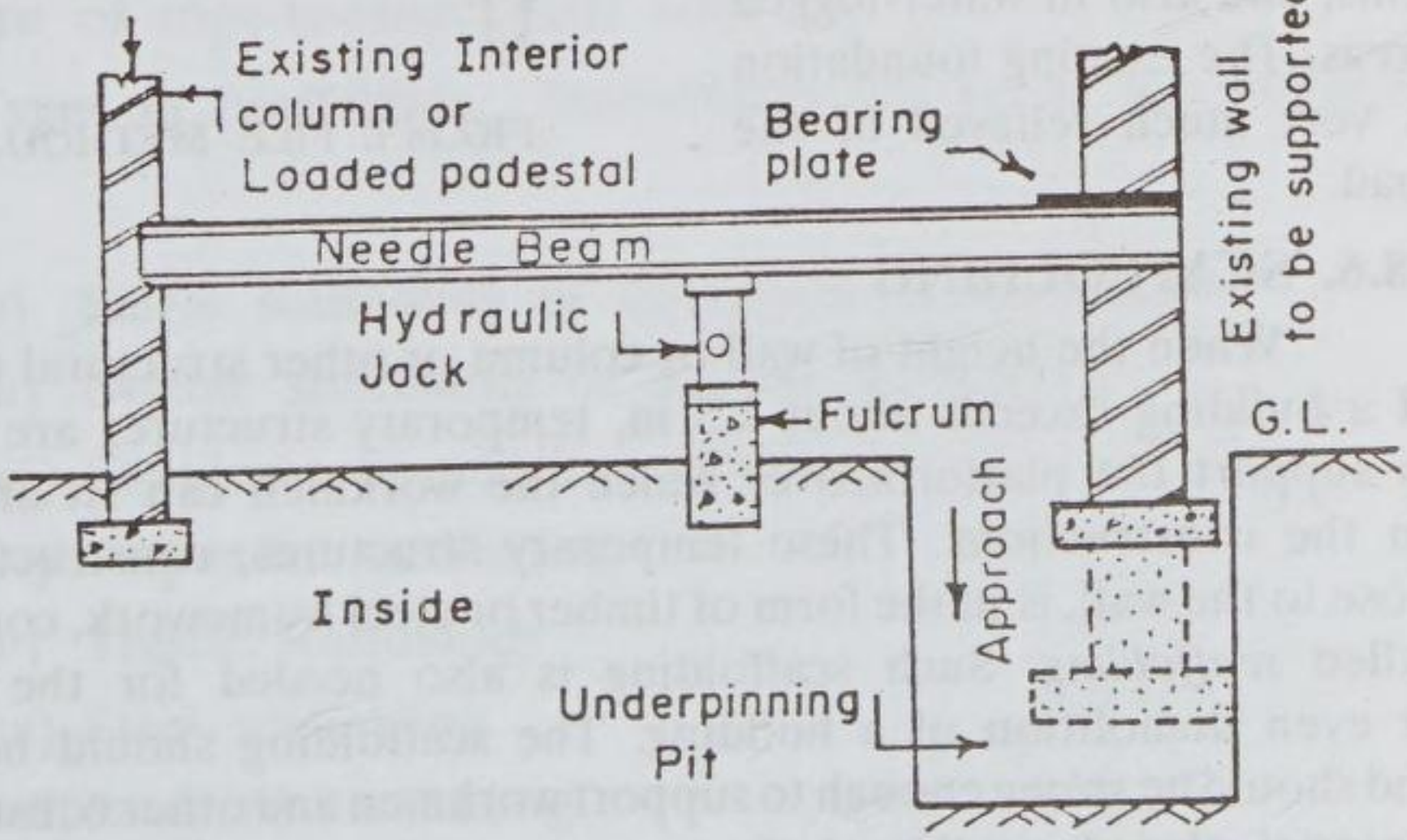


FIG. 18.6. PIT METHOD WITH CANTILEVER NEEDLE.

The following points are note-worthy in the pit method :

1. Alternate sections are taken up in the first round. The remaining intermediate sections are then taken up. Only one section should be taken at a time.

2. If the wall is long, the work is started from the middle, and is extended in both the directions.

3. If the new foundation is deeper, proper timbering of the foundation trench may be done.

4. The needle beams etc. should be removed only when the new foundation has gained strength.

5. It is desirable to do the new foundation work in concrete.

6. The needle holes etc. should be closed in masonry using cement mortar.

2.. Pile method

In this method, piles are driven at regular interval along both the sides of the wall. Generally, bore hole piles or under-reamed piles may be used. The piles are connected by concrete or steel needles, penetrating through the wall. These beams incidentally act as pile caps also. This method is very much useful in clayey soils, and also in water-logged areas. The existing foundation is very much relieved of the load.

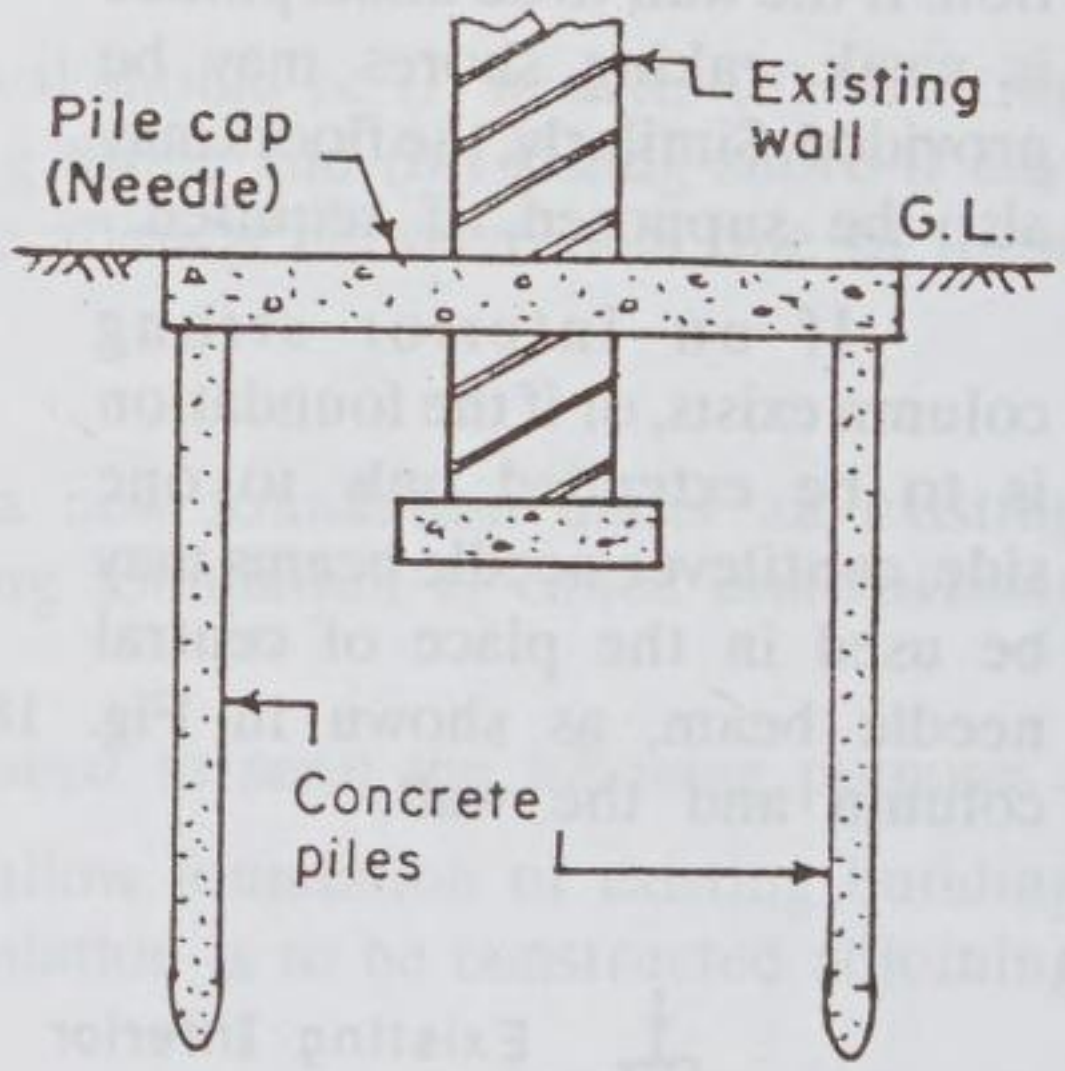


FIG.18.7. PILE METHOD.

18.6. SCAFFOLDING

When the height of wall or column or other structural member of a building exceeds about 1.5 m, temporary structures are needed to support the platform over which the workmen can sit and carry on the constructions. These temporary structures, constructed very close to the wall, is in the form of timber or steel framework, commonly called *scaffolding*. Such scaffolding is also needed for the repairs or even demolition of a building. The scaffolding should be *stable* and should be *strong* enough to support workmen and other construction material placed on the platform supported by the scaffolding. The height of the scaffolding goes on increasing as the height of construction increases.

Component parts. Scaffolding has the following components: (Refer Fig. 18.8).

(*i*) *Standards*. These are the vertical members of the frame-work, supported on the ground or drums, or embedded into the ground.

(*ii*) *Ledgers*. These are horizontal members, running parallel to the wall.

(*iii*) *Braces*. These are diagonal members fixed on standards.

(*iv*) *Putlogs*. These are transverse members, placed at right angles to the wall with one end supported on ledgers and other end on the wall.

(*v*) *Transoms*. These are those putlogs whose both ends are supported on ledgers.

(*vi*) *Bridle*. This is a member used to bridge a wall opening ; supports one end of putlog at the opening.

(*vii*) *Boarding*. These are horizontal platform to support workmen and material; these are supported on the putlogs.

(*viii*) *Guard rail*. This is a rail, provided like a ledger, at the working level.

(*ix*) *Toe board*. These are boards, placed parallel to ledgers, and supported on putlogs, to give protection at the level of working platform.

Various components or members of the scaffolding are secured by means of rope-lashings, nails bolts etc.

Types of Scaffolding Scaffoldings can be of the following types :

(*i*) Single scaffolding or brick-layers scaffolding.
(*ii*) Double scaffolding or masons scaffolding.
(*iii*) Cantilever or needle scaffolding.
(*iv*) Suspended scaffolding.
(*v*) Trestle scaffolding.
(*vi*) Steel scaffolding.
(*vii*) Patented scaffolding.

1. Single Scaffolding (Brick layer's scaffolding)

This consists of a single frame work of standards, ledgers, putlogs etc., constructed parallel to the wall at a distance of about 1.20 metres. The standards are placed at 2 to 2.5 m interval. Ledgers connect the standards, and are provided at a vertical interval of 1.2 to 1.5 m. Putlogs are placed with one end on the ledgers and other end in the hole left in the wall, at an interval of 1.2 to 1.5 m. Guards, boarding and other members are placed as shown in Fig. 18.8. Such a scaffolding is commonly used for bricklaying, and is also called *putlog scaffolding.*

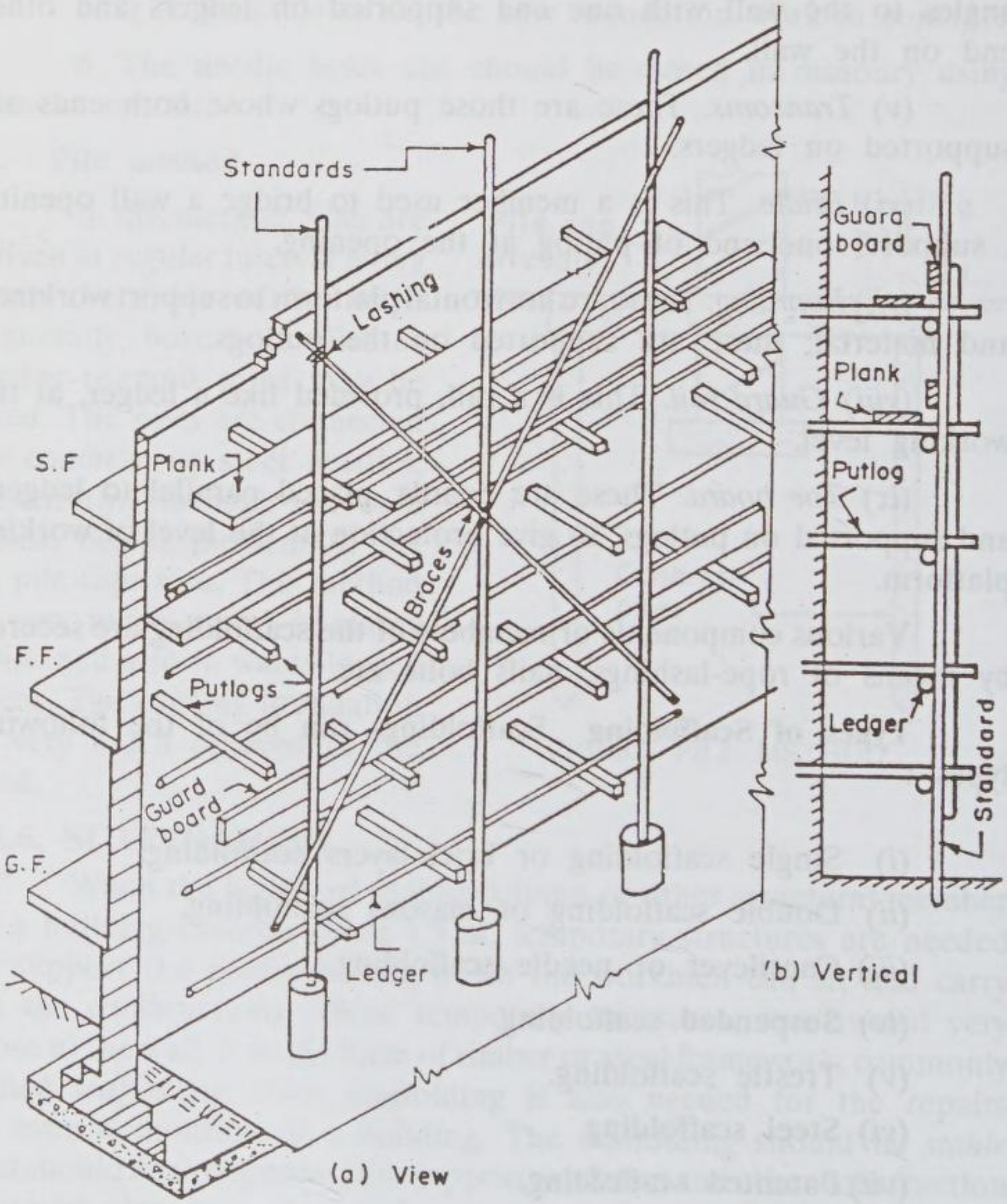


FIG. 18.8. BRICK LAYERS SCAFFOLDING.

2.. Double or Mason's Scaffolding

In stone masonry, it is very difficult to provide holes in the wall to support putlogs. In that case, a more strong scaffolding is used consisting of *two rows* of scaffolding. Each row thus forms a separate vertical frame work. The first row is placed at 20 to 30 cm away from the wall, while the other frame work is placed at 1 m distance from the first one. Putlogs are then supported on both the frames. Rakers and cross-braces are provided to make the scaffolding more strong and stable. Such a scaffolding is also called *'independent scaffolding'*.

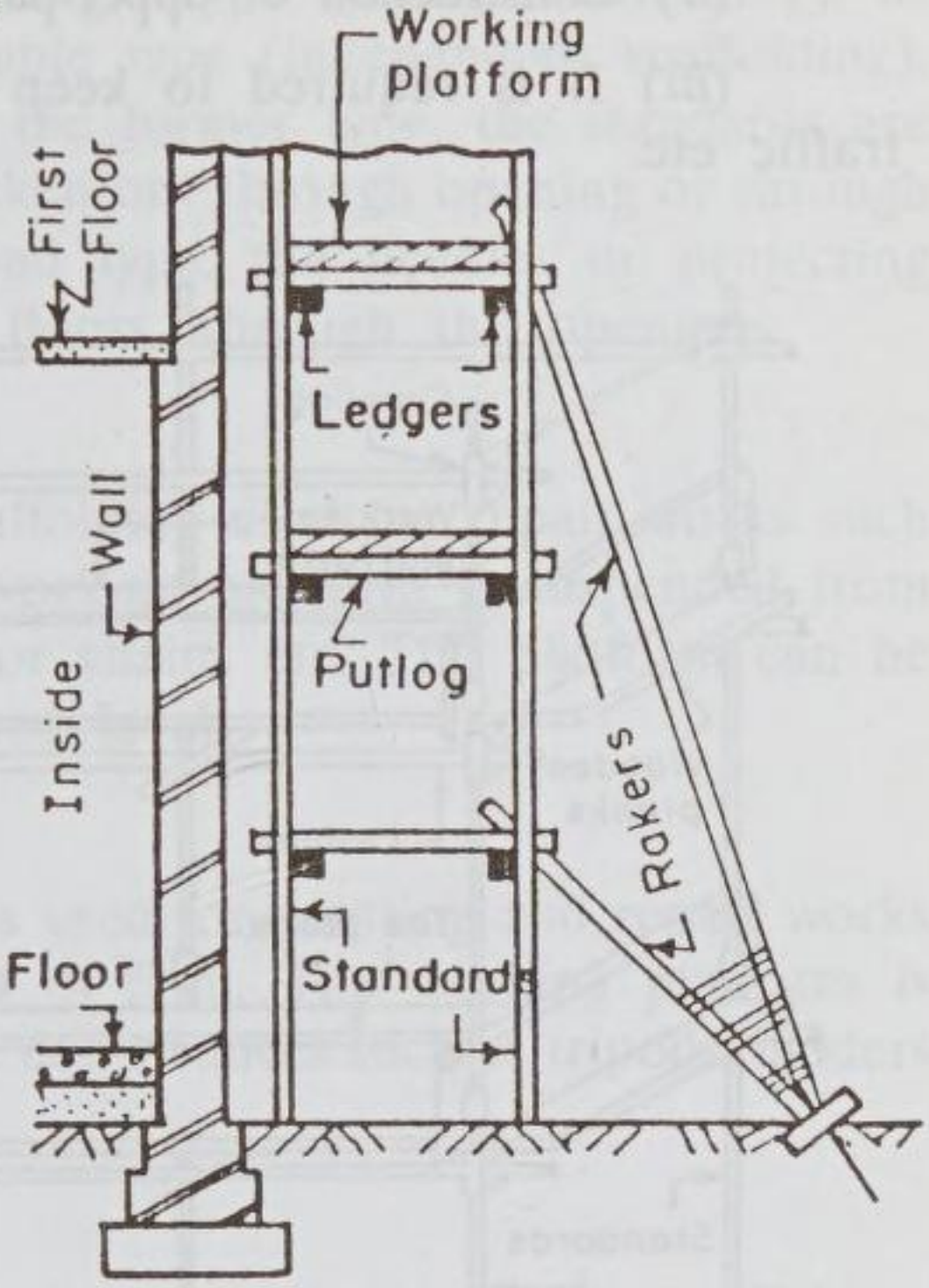


FIG. 18.9. MASON'S SCAFFOLDING.

3. Cantilever or needle scaffolding.

Cantilever scaffolding is used under the following circumstances:

(*i*) Ground is weak to support standards.

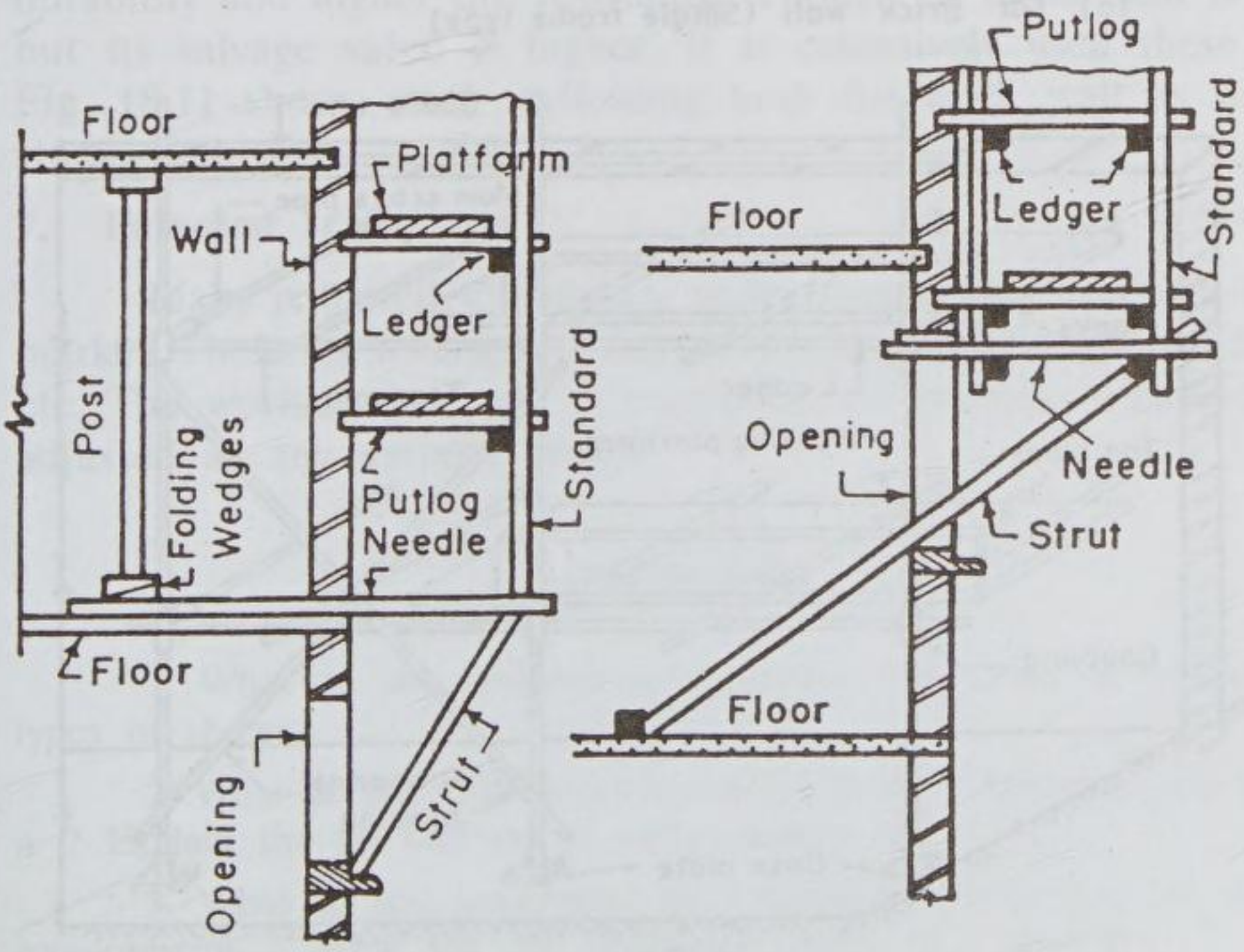
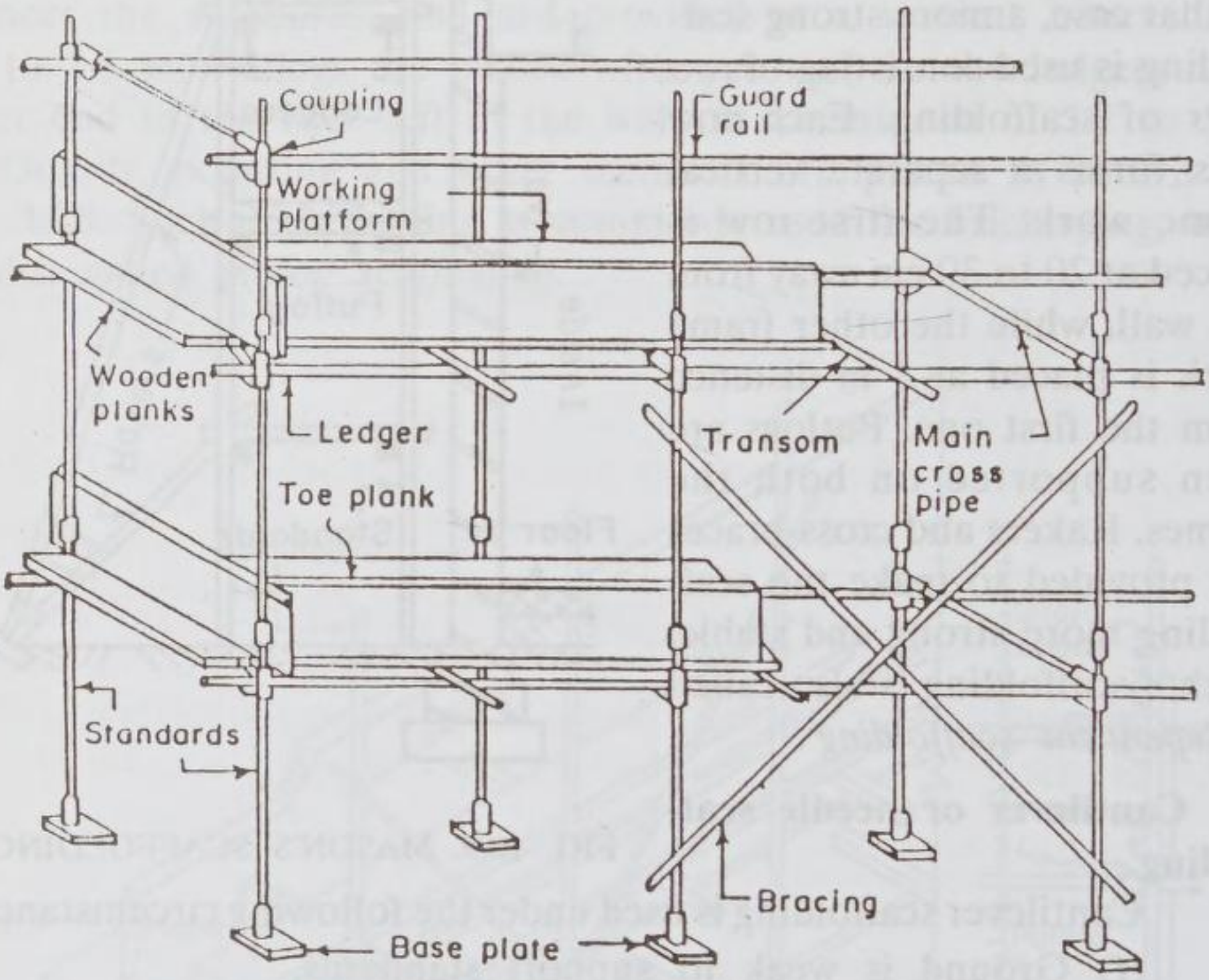


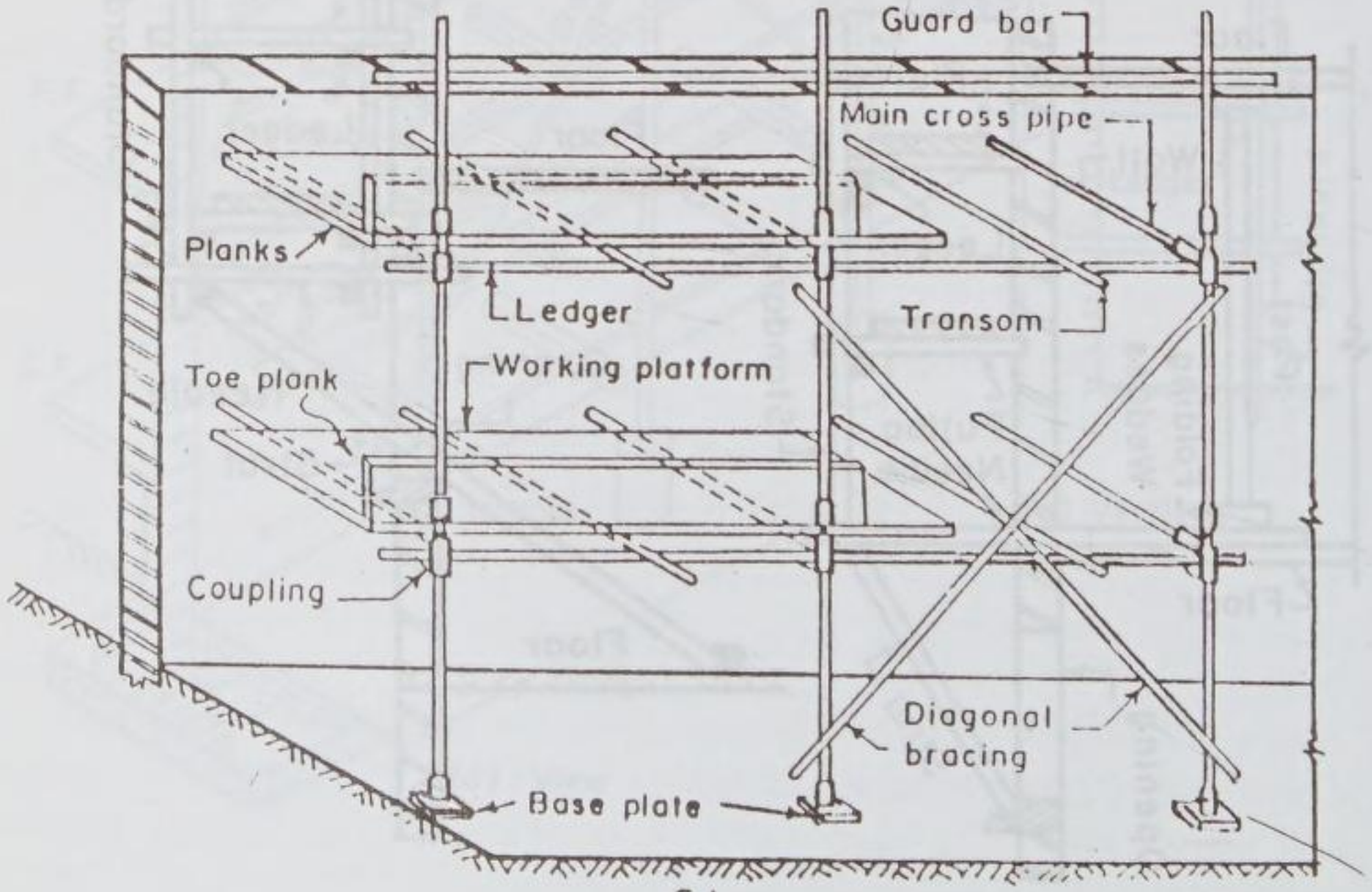
FIG. 18.10. NEEDLE SCAFFOLDING.

(*ii*) Construction of upper-part of the wall is to be carried out

(*iii*) It is required to keep the ground, near wall, free for traffic etc.



(a) For Brick wall (Single frame type)



(b) For Stone wall (Double frome type)

FIG. 18.11. STEEL SCAFFOLDING.

The scaffolding may be single type (putlog scaffolding), as shown in Fig. 18.10 (*a*) or double type (independent scaffolding), as shown in Fig. 18.10 (*b*). In the former type, the standards are supported on series of *needles* taken out through opening or through holes in the wall. In the second type, the needles or projecting beams are strutted inside the floors, through the openings.

4. Suspended scaffolding

This is a light weight scaffolding used for repair works such as pointing, painting etc. The working platform is suspended from roofs by means of wire ropes or chains etc. The platform can be raised or lowered at any desired level.

5. Trestle scaffolding

Such type of scaffolding is used for painting and repair works inside the room, upto a height of 5 m. The working platform is supported on the top of movable contrivances such as tripods, ladders etc. mounted on wheels.

6. Steel scaffolding

A steel scaffolding is practically similar to timber scaffolding except that wooden members are replaced by steel tubes and rope lashings are replaced by steel couplets or fittings. Such a scaffolding can be erected and dismantled rapidly. It has greater strength, greater durability and higher fire resistance. Though its initial cost is more but its salvage value is higher. It is extensively used these days. Fig. 18.11 shows steel scaffolding both for brick wall as well as stone wall.

7. Patented scaffolding

Many patented scaffolding, made of steel, are available in the market. These scaffoldings are equipped with special couplings, frames etc. The working platform is supported on brackets which can be adjusted at any suitable height.

PROBLEMS

1. What do you understand by shoring ? Describe in brief various types of shores.

2. What do you understand by under-pinning ? When do you require it ? Explain the pit method of under-pinning.

3. What do you understand by scaffolding ? What are the essential requirements ? What are the component parts of a scaffold?

4. Differentiate clearly between brick-layer's scaffold and mason's scaffold. Draw typical sketches.

5. Write notes on the following :

(*a*) Needle scaffolding.

(b) Dead shores.

(*c*) Flying shores.

(*d*) Steel scaffolding.

(*e*) Raking shore.

6. Compare timber scaffolding and steel scaffolding.

7. Explain how you would make a 1.2 m wide opening for a door in an existing brick wall of 30 cm thick.

19

Plastering and Pointing

19.1. PLASTERING

Plastering is the process of covering rough surfaces of walls, columns, ceilings and other building components with thin coat of *plastic mortars* to form a smooth durable surface. The coating of plastic material (*i.e.* mortar) is termed as *plaster.* Plastering on external exposed surfaces is known as *rendering.*

Objects of plastering. Plastering is done to achieve the following objects :

(1) To protect the external surfaces against penetration of rain water and other atmospheric agencies.

(2) To give smooth surface in which dust and dirt cannot lodge.

(3) To give decorative effect.

(4) To protect surfaces against vermit.

(5) To conceal inferior materials or defective workmanship.

Requirements of good plaster. The plaster material should fulfill the following requirements :

(1) It should adhere to the background, and should remain adhered during all variations in seasons and other atmospheric conditions.

(2) It should be hard and durable.

(3) It should possess good workability.

(4) It should be possible to apply it during all weather conditions.

(5) It should be cheap.

(6) It should effectively check penetration of moisture.

19.2. TYPES OF MORTARS FOR PLASTERING

The selection of type of plaster depends upon the following factors :

1. Availability of binding materials.

2. Durability requirements.
3. Finishing requirements.
4. Atmospheric conditions and variations in weather.
5. Location of surface (*i.e.* exposed surface or interior surfaces.)

Following types of *mortars* are commonly used for plastering

(*i*) Lime mortar.
(*ii*) Cement mortar.
(*iii*) Lime cement mortar.

1. Lime mortar

Lime used for plastering may be either fat lime or hydraulic lime. However, fat lime is preferred since it yields good putty after slaking. Hydraulic lime contains particles which slake very slowly as it comes in contact with atmospheric moisture ; such slaking may even continue for 6 to 8 months. If unslaked particles remain in such a plaster, blisters are formed during the process of slow slaking. Thus the plastered surface gets damaged. Hydraulic lime yields harder and stronger surface. If hydraulic lime is used for plastering, it should be ground dry with sand. It is then left for 2 to 3 weeks and then reground before use. Fat lime on the other hand, is slaked wet. The mix proportion (*i.e.* lime : sand) varies from 1 : 3 to 1 : 4 for fat lime and 1 : 2 for hydraulic or kankar lime. The binding properties of lime mortar can be improved by adding *gugal* at the rate of about 1.6 kg per cubic metre of mortar. The adhesive and tensile properties of lime mortar can further be improved by mixing *chopped hemp* at the rate of about 1 kg per cubic metre of mortar. Such a treatment prevents the formation of tensile cracks on the plastered surface.

2. Cement mortar

Cement mortar is the best mortar for external plastering work since it is practically non-absorbant. It is also preferred to lime plaster in both rooms etc., and in damp climates. Cement mortar is much stronger than lime mortar. The mix proportion (*i.e.* cement:sand) may vary from 1 : 4 to 1 : 6. Sand used for plastering should be clean, coarse and angular. Before mixing water, dry mixing is thoroughly done. When water is mixed, the mortar should be used within 30 minutes of mixing, well before initial setting takes place.

3. Lime-cement mortar

Lime-cement mortar contains properties of both the lime mortar as well as cement mortar. Cement mortar as such does not possess sufficient plasticity. Addition of lime to it imparts plasticity, resulting in smooth plastered surface. Mix proportions generally used are 1 : 1 : 6 (cement : lime : sand), 1 : 1 : 8 or 1 : 2 : 8. Generally, fat lime is used.

Table 19.1 gives the recommendations for various types of

mortar to be used in various situations.

TABLE 19.1
RECOMMENDED MORTAR MIXES

	Situation	*Composition of mortar*	*I.S. grading of lime*
1.	External Plaster in localities where rainfall is less than 500 mm per year and where subsoil water is not within 2.5 m below the ground surface :		
	(*a*) Below D.P.C.	1 cement 6 sand	—
		1 cement 2 lime 9 sand	B or C
		1 lime 2 sand	A
		1 lime 1 sand 1 surkhi	B or C
		1 lime 2 surkhi	B or C
	(*b*) Above D.P.C.	1 cement 2 lime 9 sand	B or C
		1 lime 2 sand	A
		1 lime 1 surkhi 1 sand	B or C
		1 lime 2 surkhi	B or C
2.	External plaster in localities where rain fall is more than 1300 mm per year and where subsoil water is not within 2.5 m below ground surface :		
	(*a*) Below D.P.C.	1 cement 4 sand	—
		1 cement 1 lime 6 sand	B or C
		1 lime 2 surkhi	B or C
	(*b*) Above D.P.C.	1 cement 2 lime 9 sand	B or C
		1 lime 2 sand	A
		1 lime 1 sand 1 surkhi	B or C
		1 lime 2 surkhi	B or C
3.	External plaster in localities where the subsoil water is within 2.5 m of the ground :		
	Below D.P.C.	1 cement 3 sand	—
4.	Internal plaster in all localities	1 lime 2 sand	A
		1 lime 1 surkhi 1 sand	B or C
		1 lime 2 surkhi	B or C
		1 cement 2 lime 9 sand	B or C

Note. The ratio of lime varies with % purity of lime and these ratios may be suitably adjusted depending upon local practice.

19.3. TERMINOLOGY USED IN PLASTERING WORK

1. Back ground. It is the surface to which the first coat of plaster is applied.

2. Blistering. This is the development of local swellings on the finished plastered surface, due to residual unslaked lime nodules.

3. Cracking. This is the development of one or more fissures in the plaster due to movements in the back ground or surrounding structure.

4. Crazing. This is the development of hair cracks, usually in an irregular pattern, over the finished surface.

5. Dado. This is lower part of plastered wall, where special treatment is given to make it better resistance.

6. Dots. These are small projections of plaster, laid on background for fixing of screeds etc. The size of dots may be15 cm ×15 cm.

7. Dubbing coat. This is the process of filling up hollow spaces in the solid background, before applying the main body of the plaster.

8. Finishing coat. It is the final coat of plaster. Such a coat is also known as *setting coat* or *skimming coat*.

9. Flaking. It is the process of scaling away patches of plaster of previous coat, due to lack of adhesion with the under-coat.

10. Gauging. It is the process of mixing various constituents of plaster.

11. Grinning. It is the reflection or appearance on the surface of plaster, of the pattern of joints or similar patterns in the background.

12. Grounds. These are the wooden strips fixed to the back-ground to which primary finishing may be secured.

13. Hacking. This is the process of roughening the back-ground to provide suitable bond or key for plastering.

14. Keys. These are openings or indentations or corrugations on the background or surface of under-coat, to which plaster will form mechanical bond.

15. Laitance. When freshly laid concrete or mortar is subjected to excessive trowelling a screen consisting of thin layer of fine cement particles is formed. This layer is known as laitance.

16. Peeling. This is the term applied to the dislodgment of plaster work from the background.

17. Under-coats. These are the coats of plaster applied under the finishing coat.

19.4. TOOLS FOR PLASTERING

The following tools are commonly used for plastering work:

1. Gauging trowel (Fig. 18.1 *a*). A gauging trowel is used for gauging small quantities of materials and for applying mortar to mouldings, corners etc. The end of the trowel blade may be either pointed or bull-nosed.

2. Float. A float is used for applying and spreading mortar on the surface. It is made of either metal or wood. *Metal float*,

made of thin tempered steel, is known as *laying trowel* (Fig. 18.1 *b*). The laying trowel is used for laying the plaster material and for trowelling so as to get desired finish. The blade size is generally 10 cm × 30 cm. For good work, two types of laying trowels are used. The first type having stiff plate is used for applying and trowelling the plaster, while the second type having thin plate possessing slight springing action, is used for finishing coat. The *wooden float*, commonly known as *skimming float* (Fig. 18.1 *c*) is used for the finishing coat of plaster. The size of the float varies from 10 cm × 30 cm to 11 cm × 33 cm with thickness of 10 to 12 mm. Sometimes, a *devil float*, having nail projection of about 3 mm from the surface, is used for making zig-zag lines on the plastered surface so as to form key for the subsequent coat.

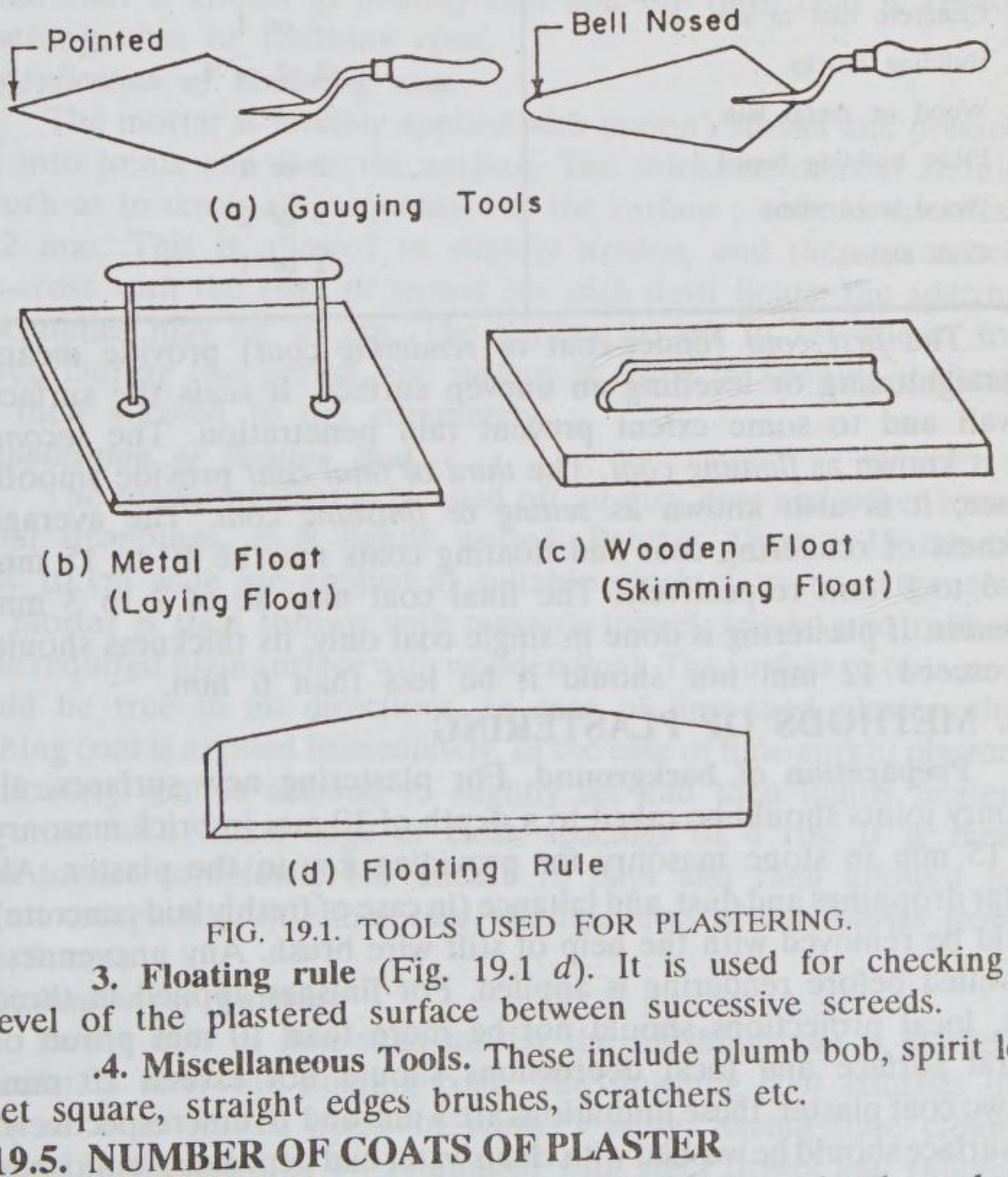


FIG. 19.1. TOOLS USED FOR PLASTERING.

3. Floating rule (Fig. 19.1 *d*). It is used for checking the level of the plastered surface between successive screeds.

.4. Miscellaneous Tools. These include plumb bob, spirit level, set square, straight edges brushes, scratchers etc.

19.5. NUMBER OF COATS OF PLASTER

The background over which plastering is to be done depend upon the type of wall construction, such as random rubble (R.R.) masonry, coarsed rubble masonry, brick masonry and cement block

work etc. Different thickness of plaster is required for different types of back-grounds. Plastering is therefore, applied in one, two or three coats. Plaster in one coat is applied only for inferior work, since it causes heavy shrinkage and consequent cracking. Generally, lime plaster is applied in three coats while cement plaster is applied in two coats. Table 19.2 gives Indian Standard recommendation for the number of coats to be applied for different type of back grounds:

TABLE 19.2
NO. OF COATS

Background	*No . of Coats*
1. Stone work	3 or 2
2. Brick work or hollow clay tiles	3, 2 or 1
3. Concrete cast in situ	2 or 1
4. Building blocks	3, 2 or 1
5. Wood or metal lath	3
6. Fibre building board	2 or 1
7. Wood wool slabs	3 or 2
8. Cork slabs	2 or 1

The *first coat* (under-coat or *rendering coat*) provide means of straightening or levelling an uneven surface. It seals the surface of wall and to some extent prevent rain penetration. The *second coat* is known as *floating coat*. The *third or final coat* provide smooth surface; it is also known as *setting or finishing coat*. The average thickness of rendering coat and floating coats may be 10 to 15 mm and 6 to 9 mm respectively. The final coat may be of 2 to 3 mm thickness. If plastering is done in single coat only, its thickness should not exceed 12 mm nor should it be less than 6 mm.

19.6. METHODS OF PLASTERING

Preparation of background. For plastering new surfaces, all masonry joints should be raked to a depth of 10 mm in brick masonry and 15 mm in stone masonry for providing key to the plaster. All mortar droppings and dust, and laitance (in case of freshly laid concrete) should be removed with the help of stiff wire brush. Any unevenness is levelled before rendering is applied. For finishes applied in three coats, local projections should not be more than 10 mm proud of general surface and local depressions should not exceed 20 mm. For two coat plaster, these limitations are 5 mm and 10 mm respectively. The surface should be washed with clean water and kept damp uniformly to produce optimum suction. In no case the surface should be kept soaked with water so as to cause sliding of mortar before it sets or kept less wet to cause strong suction which withdraws moisture

from mortar and makes it weak, porous and friable. If plaster is to be applied on old surface, all dirt, scool, oil, paint etc. should be cleaned off. Loose and crumbling plaster layer should be removed to its full thickness and the surface of the background should be exposed and joints properly raked. The surface should be washed and kept damp to obtain optimum suction.

1. LIME PLASTER

Lime plaster is applied either in three coats or in two coats. Before the application of plaster, the background is prepared as described above.

(*a*) Three-coat plaster

In the 3-coat plaster, the first coat is known as *rendering coat* second coat is known as *floating coat* and the third coat is known as setting coat or finishing coat.

1. ***Application of rendering coat***

The mortar is forcibly applied with mason's trowel and pressed well into joints and over the surface. The thickness of coat should be such as to cover all inequalities of the surface ; normal thickness is 12 mm. This is allowed to slightly harden, and then scratched criss-cross with the edge of trowel (or with devil float); the spacing of scratches may be 10 cm. The surface is left to set atleast for 7 days. During this period, the surface is cured by keeping it damp and then allowed to dry completely.

2. ***Application of floating coat***

The rendering coat is cleaned off all dirt, dust and other loose mortar droppings. It is lightly wetted. Patches 15 cm × 15 cm or strips 10 cm wide are applied at suitable spacings to act as gauges. The mortar is then thrown with mason's trowel, spread and rubbed to the required plain surface with wooden float. The surface so obtained should be true in all directions. In case of lime-sand plaster, the finishing coat is applied immediately. In the case of lime-surkhi plaster, the floating coat is allowed to slightly set and then lightly beaten criss-cross with floats edge at close spacings of 4 cm. It is then cured to set completely for atleast 10 days and then allowed to dry out completely. In either case, the thickness of coat varies from 6 to 9 mm.

3. ***Application of finishing***

In the case of lime-sand mortar the finishing coat is applied immediately after the floating coat. The finishing coat consists of cream of lime (called *neeru* or *plaster's putty*, having lime cream and sand in the ratio of 4 : 1) applied with steel trowel and rubbed and finished smooth. The rubbing is continued till it is quite dry. It is left for 1 day, and then curing is done for atleast 7 days. In the case of lime-surkhi mortar, the finishing coat is applied 7

days after the floating coat, after cleaning the surface of all dirt, dust and mortar droppings and after fully wetting the surface of previous coat. The finishing coat is rubbed hard and finished smooth.

(*b*) Two-coat plaster

In the case of two-coat plaster, the *rendering coat* is a combination of the rendering floating coats of 'three-coat plaster' and is done under one continuous operation except that the scratching of rendering coat, as specified in the three-coat plaster, is not done. The total thickness may be about 12 mm. The finishing is then applied in a manner similar to the three-coat plaster.

2. CEMENT PLASTER AND CEMENT-LIME PLASTER

Cement plaster is applied either in two coats or in three coats, the former being more common. For inferior work, single coat plaster is sometimes provided.

(*a*) **Two-coat plaster.** The following procedure is adopted :

1. The background is prepared by racking the joint to a depth of 20 mm, cleaning the surface and well-watering it.

2. If the surface to be plastered is very uneven, a preliminary coat is applied to fill up the hollows, before the first coat.

3. The *first coat* or *rendering coat* of plaster is applied, the thickness being equal to the specified thickness of plaster less 2 to 3 mm. In order to maintain uniform thickness of plaster, *screeds* are formed of plaster on wall surface by fixing dots of 15 cm × 15 cm size. Two dots are so formed in vertical line, at a distance of about 2 m, and are plumbed by means of a plumb bob. A vertical strip of mortar, known as screed, is then formed. A number of such vertical screeds are formed at suitable spacing. Cement mortar is then applied on the surface between the successive screeds and the surface is properly finished.

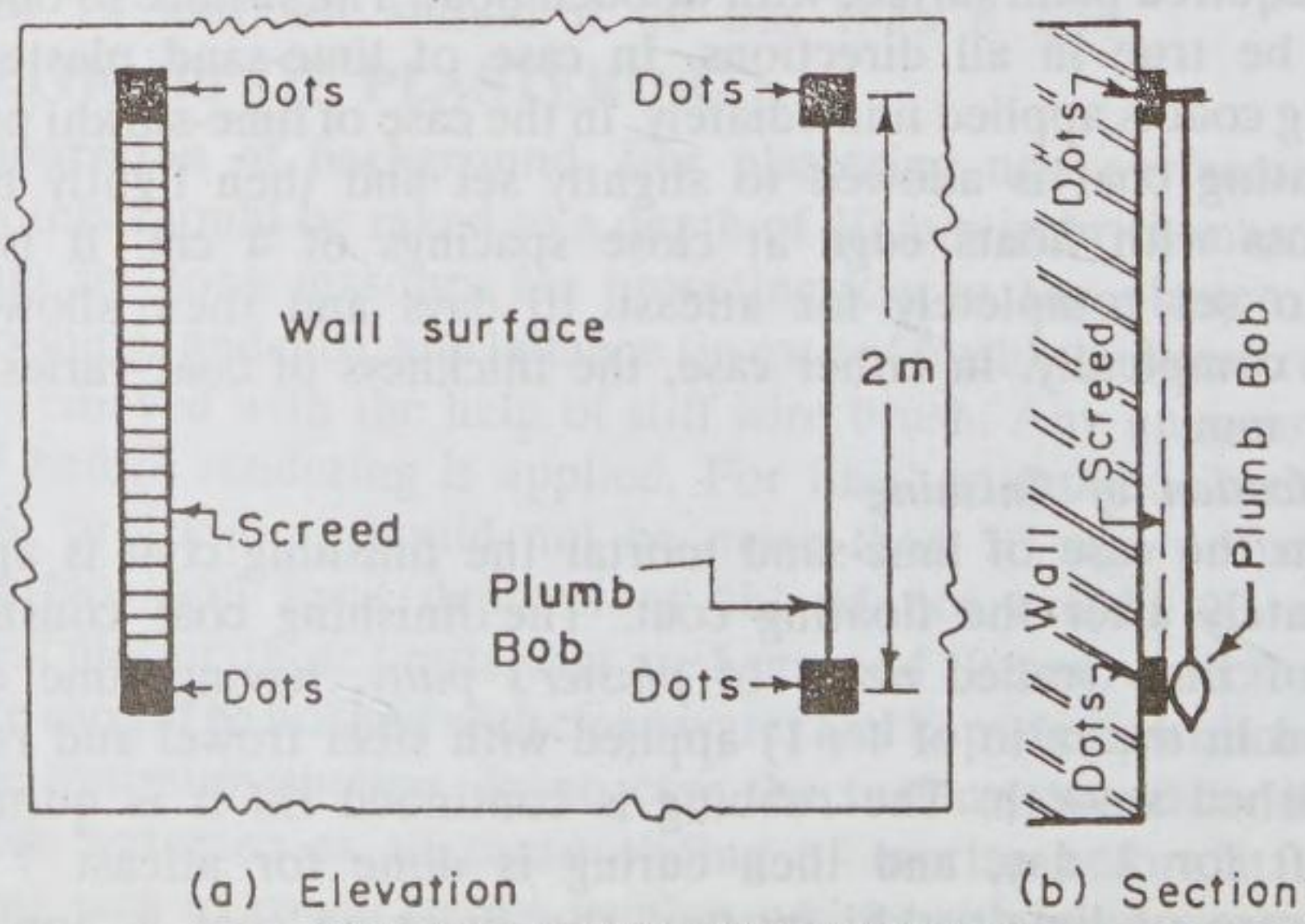


FIG. 19.2. DOTS AND SCREEDS.

4. Before rendering hardens, it is suitably worked to provide mechanical key for the final or finishing coat. The rendering coat is trovelled hard forcing mortar into joints and over the surface. The rendering coat is kept wet for at least 2 days, and then allowed to dry completely.

5. The thickness of final or *finishing coat* may vary between 2 and 3 mm. Before applying the final coat, the rendering coat is damped evenly. The final coat is applied with wooden floats to a true even surface and finished with steel trowels. As far as possible, the finishing coat should be applied starting from top towards bottom and completed in one operation to eliminate joining marks.

(*b*) **Three-coat plaster.** The procedure for applying three-coat plaster is similar to the two-coat plaster except that an intermediate coat, known as *floating coat* is applied. The purpose of this coat of plaster is to bring the plaster to an even surface. The thickness of rendering coat, floating coat and finishing coat are kept 9 to 10 mm, 6 to 9 mm and 2 to 3 mm respectively. The rendering coat is made rough. The floating coat is applied about 4 to 7 days after applying the first coat. The finishing coat may be applied about 6 hours after the application of floating coat.

(*c*) **Single-coat plaster**. This is used only in inferior quality work. It is applied similarly as two coat plaster except that the rendering coat, as applied for two-coat plaster, is finished off immediately after it has sufficiently hardened.

19.7. PLASTER ON LATH

Laths are adopted to provide foundation for plaster work. Laths are also provided for plastering thin partition walls and for plastering ceilings. Laths may be of two types : (*i*) wooden laths and (*ii*) metal laths.

Wooden laths used for plastering over wooden partition walls and ceilings, are in the form of well-seasoned wooden strips 25 mm wide and 1 to 1.2 m long. These strips are fixed in parallel lines with clear spacing of 10 mm, and secured to the surface with galvanised iron nails.

Metal laths are available under various patent names. The plain expanded metal lath (Exmat) is commonly used. Metal laths are fixed to the surface by G.I. staples. In case of concrete or masonry surface, wooden plugs have to be embedded for fixing the lath.

After fixing the lath, the surface is plastered, usually in three coats. Cement mortar is used.

19.8. TYPES OF PLASTER FINISHES

Plastered surface may be finished in the following varieties:

1. **Smooth cast finish.** In this finish, smooth, levelled surface

is obtained. The mortar for the finish may be made of cement and fine sand in the ratio of 1 : 3. Mortar is applied with the help of wooden float. Steel floats are not recommended for external renderings since they give a very smooth finish which is liable to cracking and crazing under exposure to atmospheric conditions.

2. Sand faced finish. This is obtained by plastering in two coats. The first coat is applied in 1 : 4 cement sand mortar for 12 mm thickness. It is provided with zig-zag lines. After curing it for 7 days, the second coat is applied in the thickness of 8 mm. The mortar for the second coat is prepared from cement sand mix ratio 1 : 1. The sand for this is perfectly screened so that uniform size is obtained. Sponge is used in the second coat when it is still wet. The surface of final coat is finished by rubbing clean and washed sand of uniform size by means of wooden float. This results in the surface having sand grains of equal and uniform density.

3. Rough cast finish or spatter dash finish. In this, the mortar for the final coat contains fine sand as well as coarse aggregate in the ratio of 1 : $1\frac{1}{2}$: 3 (cement : sand : aggregate). The coarse aggregate may vary from 3 mm to 12 mm in size. The mortar is dashed against the prepared plastered surface by means of large trowel. The surface is then roughly finished using a wooden float. Such a finish is water proof, durable and resistant to racking and crazing, and may be used for external renderings.

4. Pebble dash or dry dash finish. In this the final coat, having cement : sand mix proportion of 1 : 3 is applied in 12 mm thickness. Clean pebbles of size varying from 10 to 20 mm size are then dashed against the surface, so that they are held in position. The pebbles may be lightly pressed into the mortar, with the help of wooden float.

5. Depeter finish. This is similar to pebble dash finish in which the 12 mm coat is applied and while it is still wet, the pieces of gravel or flint are pressed with hand on the surface. Flints of different colours may be used to obtain beautiful patterns.

6. Scrapped finish. In this, the final coat of 6 to 12 mm thick is applied and after it has stiffened for few hours, the surface is scrapped in patterns for a depth of 3 mm. For scrapping, steel straight edge, old saw blade or such other tool may be used. Such scrapped surface is less liable to cracks.

7. Textured finish. This is used with *stucco plastering*. Ornamental patterns or textured surfaces are made on the final coat of stucco plastering, by working with suitable tools.

19.9. SPECIAL MATERIALS USED IN PLASTERING

Special materials are used in plastering or over the plastered surface to meet some specific requirements of the finished surface, such as increased durability, better or attractive appearance, fire proofing, heat insulation, sound insulation etc. Following are the usual special materials used for plastered surfaces.

1. Acoustic plaster. This contains gypsum mixtures applied as final coat in finishing the plastered surface. Such a coat undergoes chemical reaction resulting in production of gas bubbles and consequent formation of tiny openings in the coat. These honey-combed minute openings absorb sound. Such plaster is useful in the interior walls of halls, auditoriums etc. The plaster is applied in two coats each of 6 mm thickness, using wooden float.

2. Asbestos marble plaster. This plaster is made of cement, asbestos and finely crushed marble, imparting marble like finish.

3. Barium plaster. It is made from cement, sand and barium sulphate, and is provided in X-ray rooms, to protect the persons working in it.

4. Granite silicon plaster. This plaster is used for superior type of construction, since it is quick setting and possess highly elastic properties which eliminate cracks.

5. Gypsum plaster (plaster of Paris) Plaster of Paris is obtained from heating finely ground gypsum heated at 160° to 170°C. It harden within 3 to 4 minutes of adding water. To extend the setting time suitable retarders are used. Plaster of Paris is generally used in combination with lime, for ornamental work, and for repairing holes and cracks. Gypsum plaster has the following properties :

(*i*) It is fire-resisting, and hence can be effectively used on timber and metal components of buildings.
(*ii*) It is light weight.
(*iii*) It has sound insulating properties.
(*iv*) It is highly useful for ornamental work.
(*v*) It has good adhesion to fibrous materials.
(*vi*) It sets with little change in volume. Thus there is no shrinkage on drying.

However, gypsum plaster is soluble in water, hence it can be used only for interior work.

6. Kenee's cement plaster. Kenee's cement is obtained by the calcinating plaster of Paris with alum. This is very hard and sets in few days, taking white, glass-like polish. It is used for situations such as angles, skirtings etc. Because of its polishing characteristics,

it is also useful for ornamental work and decorative plastering.

7. Martin's cement plaster. Martin's cement is obtained when pearl ash is calcined with Plaster of Paris. It has quick setting properties, and forms a white hard surface on drying. It is used for internal finishing work.

8. Parian cement plaster. Parian cement is obtained when borax is calcined with Plaster of Paris. Like Kenee's cement, it is also used for interior work. However it is cheaper than Kenee's cement.

9. Scagliola plaster. Scagliola is obtained by dissolving Kenee's cement and colouring pigments in glue. It is used for plastering pilasters, panels, columns etc. It appears like marble.

10. Sirapite plaster. Sirapite is obtained when plaster of Paris is slaked in petroleum. It is quick setting and fire resisting. It produces white hard surface on drying.

11. Snowcrete and colourcrete cements. These are the trade names given to white and coloured cement respectively. These are used on external walls to create good appearance.

12. Thistle hardwall. It is a product of high grade gypsum. It sets rapidly and produces excellent finish. It is used for interior work.

19.10. DEFECTS IN PLASTERING

The following defects may arise in plaster work :

1. Blistering of plastered surface. This is the formation of small patches of plaster swelling out beyond the plastered surface, arising out of late slaking of lime particles in the plaster.

2. Cracking. Cracking consists of formation of cracks or fissures in the plaster work resulting from the following reasons :

(*i*) Imperfect preparation of background.
(*ii*) Structural defects in building.
(*iii*) Discontinuity of surface.
(*iv*) Movements in the background due to its thermal expansion or rapid drying.
(*v*) Movements in the plaster surface itself, either due to expansion (in case of gypsum plaster) or shrinkage (in case of lime-sand plaster).
(*vi*) Excessive shrinkage due to application of thick coat.
(*vii*) Faulty workmanship.

3. Crazing. It is the formation of a series of hair cracks on plastered surface, due to same reasons which cause cracking.

4. Efforescene. It is the whitish crystalline substance which appears on the surface due to presence of salts in plaster-making materials as well as building materials like bricks, sand, cement etc. and even water. This gives a very bad appearance. It affects the adhesion of paint with wall surface. Efforescene can be removed to some extent by dry brushing and washing the surface repeatedly.

5. Flaking.. It is the formation of very loose mass of plastered surface, due to poor bond between successive coats.

6. Peeling. It is the complete dislocation of some portion of plastered surface, resulting in the formation of a patch. This also results from imperfect bond.

7. Popping. It is the formation of conical hole in the plastered surface due to presence of some particles which expand on setting.

8. Rust stains. These are sometimes formed when plaster is applied on metal laths.

9. Uneven surface. This is obtained purely due to poor workmanship.

19.11. POINTING

The term pointing is applied to the finishing of mortar joints in masonry. In exposed masonry, joints are considered to be the weakest and most vulnerable spots from which rain water or dampness can enter. Pointing consists of raking the joints to a depth of 10 to 20 mm and filling it with better quality mortar in desired shape.

Mortar. pointing is done with the following mortar mixes :

(*i*) Lime mortar 1 : 2 mix (1 lime : 2 sand or surkhi)

(*ii*) Cement mortar 1 : 3 mix (1 cement : 3 sand)

The mortar for lime pointing is made with fat lime, by grinding it with sand or surkhi in a mortar mill.

Preparation of surface

(*i*) *New work.* All the joints are raked down to a depth of 20 mm while the mortar is still soft. The surface and joints are then cleaned and thoroughly wetted.

(*ii*) *Old work.* All loose pointing and superfluous mortar on the surface and in the joints are removed. The joints and surface are cleaned, and then thoroughly wetted.

Method of pointing

After preparing the surface and cleaning and wetting the joints as desired above, mortar is carefully placed in desired shape in these joints. A small trowel is used for placing the mortar in the joint: the mortar is pressed to bring perfect contact between the old interior

mortar of the joint and new mortar. Care should be taken to see that in case of ashlar and brick-work with Ist class bricks, the mortar does not cover face edges. The pointed surface is kept wet for atleast

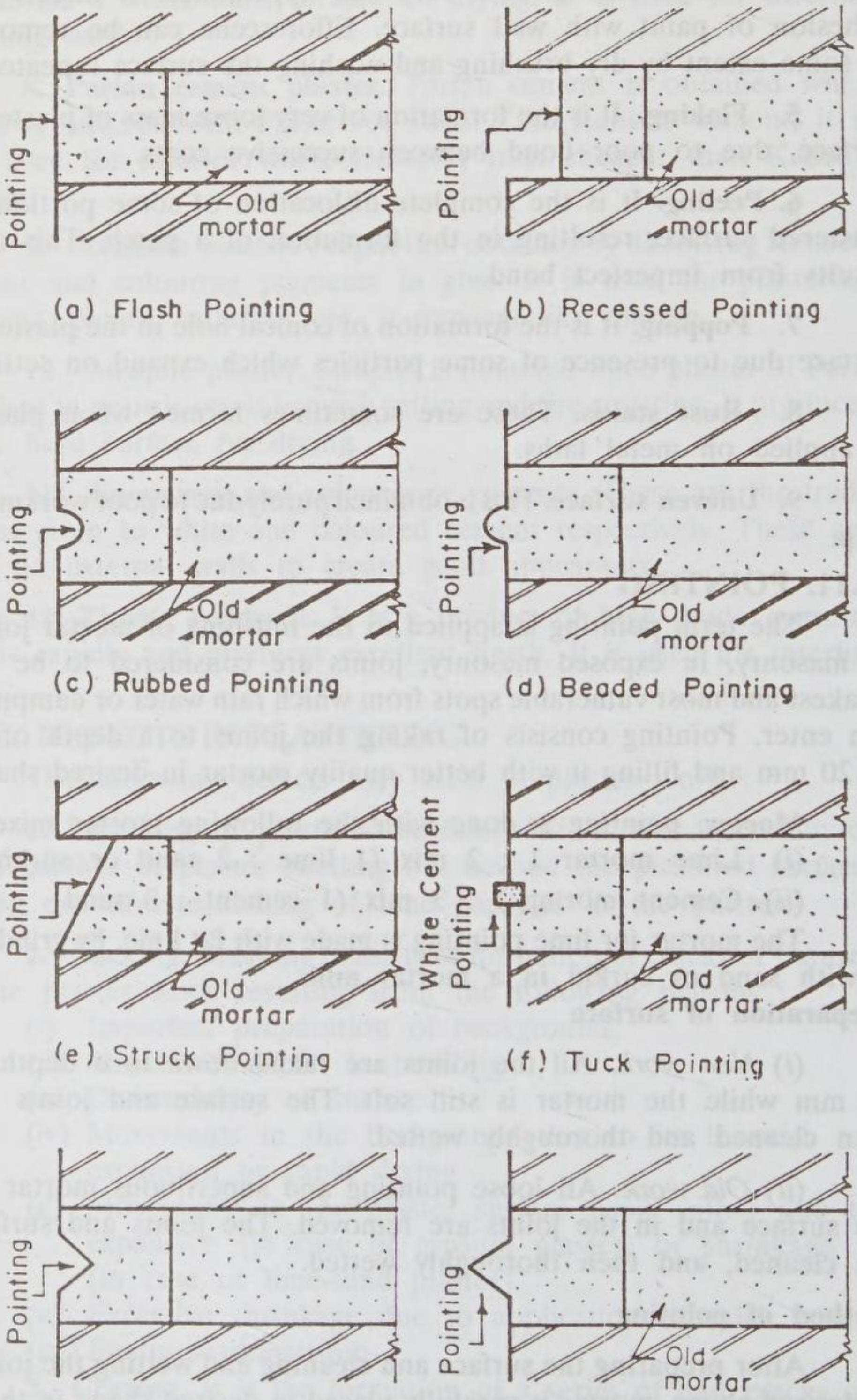


FIG. 19.3. TYPE OF POINTINGS.

a week or till it sets after application.

Types of pointings

Pointing is carried out in the following common shapes :

1. *Flush pointing* (Fig. 19.3 *a*)

This type of pointing is formed by pressing mortar in the raked joint and by finishing off flush with the edge of masonry units. The edges are neatly trimmed with trowel and straight edge. It does not give good appearance. However, the pointing is more durable since it does not provide any space for the accumulation of dust, water, etc. Due to this reason, flush pointing is extensively used.

2. *Recessed pointing* (Fig. 19.3 *b*)

The pointing is done by pressing the mortar back from the edges by 5 mm or more. The face of the pointing is kept vertical, by a suitable tool. The pointing gives very good appearance.

3. *Rubbed, keyed or grooved pointing* (Fig. 19.3 *c*)

This pointing is a modification of flush pointing by forming a groove at its mid height, by a pointing tool. It gives better appearance.

4. *Beaded pointing* (Fig. 19.3 *d*)

This is the special type of pointing formed by a steel or ironed with a concave edge. It gives good appearance, but is liable to damage easily.

5. *Struck pointing* (Fig. 19.3 *e*)

This is a modification of flush pointing in which the face of the pointing is kept inclined, with its upper edge pressed inside the face by 10 mm. This pointing drains water easily.

6. *Tuck pointing* (Fig. 19.3 *f*)

The pointing is formed by first pressing the mortar in the racked joint and finishing flush with the face. While the pressed mortar is green, groove or narrow channel, having 5 mm width and 3 mm depth is cut in the centre of the groove. This groove is then filled in or *tucked in* with white cement putty, kept projecting beyond the face of the joint by 3 mm. If projection is done in mortar, it is called *Bastard pointing* or *half tuck pointing.*

7. *V-pointing* (Fig. 19.3 *g*)

This pointing is formed by forming V-groove in the flush-finishing face.

8. *Weathered pointing* (Fig. 19.3 *h*)

This pointing is made by making a projection in the form of V- shape.

PROBLEMS

1. (*a*) Explain in brief the objects of plastering and pointing.
   (*b*) What are the requirements of good plaster ?
   (*c*) Write a note on 'mortars' required for plastering and pointing.
2. (*a*) What do you understand by preparation of back ground for (*i*) plastering, (*ii*) pointing ?
   (*b*) Write a note on number of coats used in plastering.
3. Explain various types of plaster finishes.
4. Explain the method of three-coat lime plaster.
5. Explain the method of two-coat lime plaster.
6. Write a note on various types of special materials used in plastering.
7. Write a note on various defects in plastering.
8. (*a*) Explain the method of pointing.
   (*b*) Describe various types of pointings.

20

Painting, Distempering and White-Washing

20.1. PAINTS AND PAINTING

Paints are liquid compositions of pigments and binders which when applied to the surface in thin coats dry to form a solid film to impart the surface a decorative finish, apart from giving protection to the base material (*i.e.*, concrete, masonry and plaster surfaces) from weathering, corrosion and other chemical and biological attacks. Paints preserve timber structures against warping and decay. Most of the metals corrode if not painted at suitable interval. Painting on surfaces impart decoration, sanitation and improved illumination.

Calcareous surfaces, like lime and cement plastered surfaces, are highly alkaline in the initial stages, they retain large quantities of water during construction and it takes long time for the greater part of the water to evaporate even when the atmospheric conditions are favourable. Therefore, in applying a paint system on these surfaces, it is essential to take cognisance of the stored up moisture and also the alkalinity of the surfaces. These surfaces are porous and present problems, such as variable suction, surface imperfections, growth of moulds, mosses, lichens and algae. As each of these have adverse effect on most of the surface coating materials, finishing of these surfaces need special care.

20.2. CHARACTERISTICS OF AN IDEAL PAINT

An ideal paint should possess the following characteristics :

(1) Paint should form hard and durable surface.

(2) It should give attractive appearance.

(3) It should be cheap and readily available.

(4) It should be such that it can be applied easily to the surfaces.

(5) It should have good spreading quality, so as to cover maximum area in minimum quantity.
(6) It should dry in reasonable time.
(7) It should not show hair cracks on drying.
(8) It should form film of uniform colour, on drying.
(9) It should be stable for a longer period.
(10) It should not be affected by atmospheric agencies.

For efficient planning and execution of painting work on plaster surfaces, the following informations should collected :

(*a*) Type of concrete, masonry or plaster surfaces to be painted, the type and nature of previous treatment, if any.
(*b*) Situations of use, namely, external finish or an internal finish; and the extent to which the surface will be exposed to weather and rain ; and
(*c*) In the case of new plastered surfaces, the nature of backing, the type of plaster undercoat and finish, the approximate date of completion of the plaster work in individual rooms ; and any addition of lime to the plaster finishing coat should be noted.

20.3 CONSTITUENTS OF A PAINT

A paint generally is made up of the following constituents :
(*i*) A base.
(*ii*) A vehicle or carrier.
(*iii*) A drier.
(*iv*) A colouring pigment.
(*v*) A solvent or thinner.

1. Base.

A base is a solid substance in a form of fine powder, forming the bulk of a paint. It is generally a metallic oxide. The type of base determines the character of the paint and imparts durability to the surface painted. Various bases commonly used are : (*i*) White lead, (*ii*) Red lead, (*iii*) Oxide of Zinc (Zinc white), (iv) Oxide of iron, (*v*) Titanium white, (vi) Antimony white, (*vii*) Aluminium powder, and (*viii*) Lithophone. For a detailed description and characteristics of these, reference may be made to Author's book 'Building Materials'. A base in a paint provides of opaque coating to hide the surface to be painted.

2. Vehicle or carrier or binder

These are liquid substances which hold the different ingradients of a paint in liquid suspension. The carrier or vehicle makes it possible to spread the paint evenly on the surface. The vehicles generally in use are : (*i*) various forms of linseed oils (such as raw linseed oil, boiled linseed oil, pale boiled linseed oils, double boiled

linseed oil and stand oil, (*ii*) tug oil, and (*iii*) poppy oil, and (*iv*) nut oil. Raw linseed oil is thin, but it takes a long time to dry. Boiled linseed oil is thicker. For delicate work, however, only raw linseed oil is used along with driers and poppy and nut oil. It is used for interior work. Double boiled linseed oil dries very quickly and is suitable for external work. It requires thinning agent like turpentine. *Tug oil* is used for preparing paints of superior quality. Colours in poppy oil last longer.

3. Drier

Driers are used to accelerate the process of drying and hardening, by extracting oxygen from the atmosphere and transferring it to the vehicle. However, driers reduces the elasticity of the paint; they should not be used in the final coat. Driers may be in the form of soluble driers or paste driers. Liquid driers are finely ground compounds of metals such as cobalt, lead, manganese dissolved in a volatile liquid. Paste driers consist of compounds of the above metals mixed with large percentages of inert fillers such as barytes, whiting etc., and then ground in linseed oil. The inert fillers serve the following purposes : (*i*) reduce the cost of paint, (*ii*) improve durability, (*iii*) modify the weight, and (*iv*) prevent shrinkage and cracking. However, these are termed as adultrants, and their weight should not exceed one-fourth the weight of the base. Litharge (PbO), red lead (Pb_3O_4) and sulphates of zinc and manganese are also used as driers. Litharge is most common in use but in general lead drier should not be used with zinc paints.

4. Colouring pigment

Colouring pigments are added to the base to have different desired colours. Pigments can be divided into the following divisions

(*i*) *Natural colours*	: Ochres, umbers, iron oxides.
(*ii*) *Calcined colours*	: Lamp black, Indian red, carbon black, red lead.
(*iii*) *Precipitates*	: Prussian blue, chrome green, chrome yellow.
(*iv*) *Lakes*	: Prepared by discolouring barytes or china clay with the help of suitable dyes.
(*v*) *Metal powders*	: Powders of aluminium, bronze, copper, zinc, etc.

The desired shade or tint of the paint may be obtained by using single or combination of the following colouring pigments.:

Tint	*Pigment*
1. Black	: Lamp black ; carbon black ; bone black ; graphite, vegetable black; ivory black.

2. Blue : Indigo ; Prussian blue ; cobalt blue ; ultramarine.
3. Brown : Burnt umber, raw umber, burnt seinna, vandyke brown.
4. Green : Paris green : chrome green; green earth; virdigris copper sulphate.
5. Red : Indian red; venitean red; vermillion red; carmine; red lead.
6. Yellow : Chrome yellow; raw seinna; yellow occhre; zinc chrome.

The concentration of pigment in a paint is denoted by *pigment volume concentration number* (P.V.C.N.) defined by the equation.

$$\text{P.V.C.N.} = \frac{V_1}{V_1 + V_2}$$

where V_1 = volume of pigment in the paint.
V_2 = Volume of non-volatile vehicle or carrier in the paint.

The durability and gloss of a paint is inversely proportional to the value of P.V.C.N. The following table gives P.V.C.N. for paints used for various purposes :

P.V.C.N. range	*Type of paint*
25 to 40	Paint for prime coat on metals.
35 to 40	Paint for prime coat on timber.
28 to 40	Paint for exterior surfaces of buildings.
35 to 45	Semi-gloss paint.
50 to 75	Faint paint.

5. Solvents or thinners

Solvents are added to the paint to make it thin so that it can be easily applied on surfaces. It also helps the paint in penetrating through the porous surface of the background. The thinning agent commonly used is the spirit of terpentine. Other solvents contain some part of spirit of terpentine, and therefore inferior. Thinner reduces the gloss of the paint. Terpentine oil is affected by weather; hence minimum quantity of thinner should be used for painting external surfaces. Following is the list of thinners for various types of paints :

	Type of paint	*Thinner*
1.	Oil paints	(*i*) Spirit of terpentine. (*ii*) Naphtha, (*iii*) Benzine
2.	Spirit liquors	Alcohol.
3.	Cellulose paints	Ethyl amyl acetate.
4.	Distempers	Water.

20.4. CLASSIFICATION AND TYPES OF PAINTS

Standardising the classification of paints is difficult in view of the large number of variations in each of the constituents, but a simple classification based on the media or binder, and on the basis of its ultimate use and performance is given here.

(*a*) **Classification based on binders**

(*i*) Oil paints
(*ii*) Paints based on non-oil resins.
(*iii*) Cellulose paints.
(*iv*) Water based paints.
(*v*) Miscellaneous paints.

(*b*) **Classification based on ultimate use**

(*i*) General purpose paints, including primers, under-coat paint and finishing coat paints
(*ii*) Acid and alkali resistant paints
(*iii*) Fire resistant paints
(*iv*) Fungicidal paints
(*v*) Miscellaneous paints. such as fire resistant paints, anti-condensation paint etc.

(*c*) **Mixed classification : types of paints**

(*i*) Aluminium paint.
(*ii*) Anti-corrosive paints
(*iii*) Asbestos paints
(*iv*) Bituminous paints
(*v*) Bronze paints
(*vi*) Casein paints
(*vii*) Cellulose paints
(*viii*) Cement-based paints
(*ix*) Colloidal paints
(*x*) Emulsion paints
(*xi*) Enamel paints
(*xii*) Graphite paints
(*xiii*) Inodorous paints
(*xiv*) Oil paints
(*xv*) Plastic paints
(*xvi*) Silicate paints
(*xvii*) Synthetic rubber paints.

1. Aluminium paints. It consists of finely ground aluminium suspended in either quick-drying spirit varnish or slow-drying oil varnish, as per actual requirements. A thin metallic film of aluminium is formed when the spirit or oil evaporates. It is used for painting wood work or metal surfaces. This paint has following advantages: (*i*) Weather resistant, (*ii*) Water proof, (*iii*) Highly heat reflective, (*iv*) Corrosion resistant, (*v*) High electrical resistance, (*vi*) High covering

capacity, (*vii*) Visibility in darkness, (*viii*) Better appearance.

2.. Anti-corrosive paints. It is used to protect metal structures against adverse effects of moisture, fumes, acids, corrosive chemical ravages of rough weather. It consists of oil and a strong drier and a colouring mixed with very fine sand. Due to this, it is cheaper than white/lead paints. It lasts longer. However, it gives black appearance. Linseed oil is generally used as vehicle.

3. Asbestos paints. This is a special-purpose paint used for painting surfaces which are exposed to acidic gases and steam, and also for patch work or stopping leakage in metal roofs. It is also used for painting gutters, pouts, flashings etc., to protect them from rusting. The paint consists of fibrous asbestos as the main ingradient.

4. Bituminous paints. These paints are prepared by dissolving asphalt, tar or mineral pitches in naphtha, petroleum or white spirit. These paints are alkali resistant and are mainly used for painting structural steel under water, and iron water mains. The paint gives black appearance, and deteriorates when exposed to the direct sun rays.

5. Bronze paints. These paints are also used for painting interior and exterior metallic surfaces. The paint consists of nitro-cellulose lacquer as vehicle and a aluminium bronze or copper bronze as pigments. Because of its high reflective property, the paint is used on radiators.

6. Caesin paints. Caesin, a protein substance extracted from milk curd, is mixed with a base consisting of white pigments, to form the paint which is available in powder or pasty form. The paint can be applied on walls, ceilings, wall boards etc. to enhance the appearance. It can be tinted in any desired shade of colour. For painting exterior surfaces, a little quantity of drying oil or varnish is added to make it weather well.

7. Cellulose paints. This paint is different from the ordinary oil paints. It is prepared from nitrogen-cotton, celluloid sheets, photographic films and amyl-acetate substitutes. The paint hardens by evaporation of thinner or solvent, while oil paints harden by oxidation. The paint gives very smooth finish which remains unaffected by hot water, smoky or acidic atmosphere etc. Due to its high cost, it is used for painting motor cars, aeroplanes etc.

8. Cement-based paints. This paint is a type of water paint in which white or coloured cement forms the base. No oil is used. It is available in powdered form, consisting of cement, pigment, accelerator and other additives; it is available in different trade names such as snow-cem etc. The paint is readily made by mixing water to the powder to obtain thick smooth paste and then diluting it to the required consistency. The paint is very much useful for painting

external surfaces, since it is water proof. For new surfaces, it is applied in three coats while for old surfaces, it is applied in two coats.

9 **Colloidal paints**. This paint does not contain any inert material. Because of its colloidal properties, it takes more time to settle. In the process of settlement, it penetrates through the surface on which it is applied.

10. Emulsion paints. This paint contains binding materials (vehicles) such as polyvinyl lacetate, styrene, alkyd resin and other synthetic resins. The vehicle imparts alkali-resistance to the paint. The paint dries quickly, within $1\frac{1}{2}$ to 2 hours. It has good workability and high durability. The principle film forming constituent of this paint is emulsified in water, so that it may be thinned with water instead of solvent. The painted surface can be washed with water. It is recommended for use on stuccoplaster, bricks and masonry surfaces which contain free alkali.

11. Enamel paints. Enamel paint contains four basic constituents — metallic oxide (white lead or zinc white), oil, petroleum spirit and resinous matter. The paint dries slowly, but on drying, it produces a hard, impervious, glossy, elastic smooth and durable film. Different types of enamel paints are available in readymade forms, in a variety of colours. The painted surface is not affected by acids, alkalies, fumes of gas, hot and cold water, steam etc. It is commonly used on doors, windows, metal grills, porches, decks, stairs, concrete stairs etc.

12. Graphite paints. This paint has black colour, and is used for painting iron surfaces which come in contact with ammonia chlorine, sulphur gases. It is also used for mines and underground structures.

13. Inodorous paints. This paint contains white lead or zinc white mixed with methylated spirit. Shellac with some quantity of linseed oil and caster oil is dissolved in methylated spirit. No turpentine is used. The paint dries very quickly, due to evaporation of methylated spirit, leaving behind a thin film of shellac.

14. Oil paints. Oil paint is an ordinary paint consisting of two principal constituents : a base and a vehicle. However, driers and colouring pigments are also added. Vehicles that are generally used in oil paints are : linseed oil (raw), boiled linseed oil, linseed oil pale boiled, tug oil etc. The base pigments generally used are white lead, red lead, zinc white, lithophone and titanium oxide. Driers commonly used are litharge, red lead, and sulphates of zinc and manganese. Oil paints are generally used in three coats : prime coat, under coat and finishing coat, each having varying composition. Oil paints are cheap, easy to apply and possess good capacity and low gloss. They are used in general for all types of surfaces such

as walls, ceilings, wood work, metal work, etc. However, oil paint should not be applied during humid and damp weather. Oil paints possess all the characteristics of a good paint, therefore, are commonly used.

15. Plastic paints. These paints contain plastic as the base which forms the main constituent of the paint. These paints have the qualities of quick drying, high covering power and decorative appearance. *Plastic emulsion paint* has become very popular these days. The emulsion, which is a liquid having fine suspended particles of a substance, is composed of a plastic compounds such as vinyl acetate and acrylate which are held in water. A litre of plastic emulsion paint, weighing about 1.4 kg, contains about 0.20 kg of binder, 0.50 kg pigments, 0.10 kg other solids and 0.60 kg water. One litre of plastic emulsion paint can cover 15 m^2 of wall surface per coat. It is applied in two coats, either with the help of a brush or a spray gun.

16. Silicate paints. A silicate paint is prepared by mixing calcined and finely ground silica with resinous substances. Silica imparts good adhesion to the paint. It forms very hard and durable surface on drying. It can withstand extreme heat. It is not affected by alkalies. The paint has no chemical action on metals.

17. Synthetic rubber paints. These paints consist of synthetic resins dissolved in appropriate solvents and mixer with suitable pigments. The paint has excellent acid, alkali and moisture resistance properties. it is little affected by rain, sunlight and other weather changes. It dries quickly, and uniform colour is maintained. It has moderate cost, and can be applied on cement concrete more and interior and exterior masonry surfaces.

20.5. PAINTING ON DIFFERENT SURFACES

1. PAINTING ON NEW WOOD WORK

The painting on new wood work is done on the following steps. For good work, 4 coats of paints are required, while for inferior work, only 2 to 3 coats are applied.

(*i*) **Preparation of surface.** For good results, wood work should be well-seasoned, and should not contain more than 15% moisture. The surface is dusted off thoroughly to remove dust, shavings, foreign matter etc. Heads of nails are punched to a depth 3 mm below the surface to be painted. Greasy spots, if any, should be removed by rubbing with piece of clean white muslin soaked in benzine or terpentine, allowed to dry, and glass papered if necessary.

(*ii*) **Knotting.** Knotting is the process of covering or killing all knots in the wood work with a substance through which the resin cannot come out or exude. Otherwise, the resin coming out of knots would damage the paint. Knotting can be done by three

methods. In the first method, called *ordinary or size knotting*, two coats are applied. The first coat consists of grounding 15 g of red lead in 2 litres of water, adding 225 g of glue and heating the solution. This coat dries in 10 minutes, and then the second coat is applied. The second coat consists of red lead ground in boiled linseed oil and thinned with turpentine oil. The second method is known as *patent* which consists of applying a coat of hot lime, leaving it for 24 hours, scrapping off the surface and then carrying out ordinary or size knotting.

(*iii*) **Priming.** After knotting, the surface is rubbed smooth with a abrasive paper. Priming consists of applying first coat of paint to fill all the pores. Priming coat creates a layer or film which provides adhesion of the paint with the surface. Usually, the ingradients of the paint are kept the same as in subsequent coats though in varying proportion. The composition of primer for *ordinary work* may be composed of 3 kg red lead, 3 kg white lead 3 litre of linseed oil or terpentine. For superior work, the following composition is recommended :

For interior work			*For exterior work*		
Red lead	= 0.25 kg	(base)	Red lead	= 0.04 kg	(base)
White lead	= 3.5 kg		White lead	= 4.5 kg	
Boil Linseed oil	= 0.5 litre	(vehicle)	Raw linseed oil	= 2.25 litr	(vehicle)
Raw Linseed oil	= 0.5 litre				
Litharge	= 0.05 kg	(drier)	Litharge	= 0.09 kg	(drier)

Generally, the priming coat is applied before fixing wood work in position.

(*iv*) **Stopping.** It is the process of rubbing down the wood surface by means of pumice stone or glass paper after prime coat is applied, and then filling up all cracks, all nail holes, dents, open joints etc., with putty. After putty dries up, the surface is rubbed again with pumice stone or glass paper. The putty is made by mixing powdered chalk in linseed oil to the consistency of a thick paste. For superior work, *hard stopping* is restored to by using one-third white lead and two-thirds ordinary putty in place of ordinary putty.

(*v*) **Under-coatings.** After stopping, second and successive coatings (known as under-coatings) are applied. The first coat is the primecoat. The under-coatings should be of the same shade as that

of the finishing coat. The under-coatings may be necessary, depending upon the quality of work desired. Sufficient time should be allowed for each coat to dry before next coat is applied. For superior work, each coat is allowed to dry, rubbed down with pumice stone or glass paper then cleaned before next coat is applied.

(*vi*) **Finishing coat.** Finishing coat is applied after the under-coat is perfectly dry. This coat is applied very carefully, by a skilled painter, so that finished surface is smooth, uniform and free from patches and bush marks.

2. RE-PAINTING OLD WORK

Before repainting old work, the old paint having cracks and blisters should be removed, by applying any one of the following solvents or paint removers :

(*i*) Applying solution containing 1 kg of caustic soda in 5 litres of water. The paint gets dissolved.

(*ii*) Applying mixture containing one part of soft soap, two parts of potash and one part of quicklime, while in hot state. After 24 hours of application, the surface is washed with hot water.

(*iii*) Applying mixture of equal parts of washing soda and quicklime to the required consistency. After 1 hour of application, the surface is washed with water.

After removing the old paint, the surface is properly cleaned and then rubbed with pumice stone or glass paper. The cleaned surface is given two or three coats of paint to obtain the desired finish.

3. PAINTING NEW IRON AND STEEL WORK

Iron and steel surfaces are painted so that rusting is prevented. Hence surface should be prepared very carefully.

(*i*) The surface is cleaned off scale and rust etc. by scrapping or brushing with steel wire brushes. Oil, grease, etc. is removed by washing the surface with petrol, benzene or lime water.

(*ii*) The cleaned surface is treated with a film of phosphoric acid. This film protects the surface from rusting and provides better adhesive surface for the paint.

(*iii*) The prime coat or first coat is then applied with a brush. The coat consists of dissolving 3 kg of red lead in 1 litre of boiled linseed oil.

(*iv*) After the prime coat has dried, two or more under-coats are applied either with a brush or with spray gun. Care should be taken to see that each successive coat is applied only after the previous coat has dried completely. The under-coat may consist of 3 kg of red oxide, dissolved in 5 litres off boiled linseed oil.

(*v*) After the under-coat has dried, the final coat of the desired

type of paint is applied. The finishing coat should present smooth finish.

4. REPAINTING OLD IRON AND STEEL WORK

Before repainting, the old surface is thoroughly cleaned by application of soap water. The grease, if any, may be removed by washing the surface with lime and water. However, if the old paint has cracked, it has to be removed by *flame-cleaning.* A flat oxy-acetylene flame is passed over the metal, burning off the old paint and loosening rust and scale. The surface is then scrapped with wire brush and washed with solution of caustic soda and fresh slaked lime. After the surface is thus prepared, painting is carried out as for the new surface.

5. PAINTING GALVANISED IRON WORK

Since paint does not easily adhere to the surface of G.I. work, some special treatment is necessary before the application of prime coat. It is better to paint the work only after exposing it to weather for about a year. However, if immediate painting is required, the surface may be treated with the following :

(*i*) Solution containing 40 gm of copper acetate in one litre of water.

(*ii*) Solution containing 13 gm each of muriatic acid, copper chloride, copper nitrate and ammonium chloride in one litre of water. This solution is sufficient to cover an area of 250 to 300 m^2.

The solution is taken in glass vessel or earthware vessel. After application of any one of the above solutions, the surface turns black. The prime coat is then applied after 12 hours. The prime coat may consist of red lead mixed with linseed oil and turpentine in equal proportions. When the prime coat dries, suitable paint may be applied.

6. PAINTING OTHER METALS

Before painting, the surface should be clean, dry, and free from dirt, grease etc. Suitable prime coat should be selected for each type of metal surface to be painted. The following prime coats are suggested :

Metal surface	*Prime coat*
(*i*) Aluminium surface	Zinc chromate
(*ii*) Zinc surface	Zinc oxide

After applying prime coat, painting is carried out exactly in the same manner as adopted for iron and steel surfaces.

7. PAINTING PLASTERED SURFACES

Newly plastered surface may contain considerable moisture. Hence painting should be resorted to only after 3 to 6 months of plastering. Calcareous surfaces to lime and cement plastered surfaces are highly alkaline because lime is liberated during hydration of

cement. Due to this, oil based paints and distempers are liable to alkali attack. Hence it is essential to apply *alkali resistant primer.* Absorption of liquid from a paint by a porous surface is known as *suction*. High suction may make the paint difficult to apply and leave the coating in an underbound condition. Uneven suction may cause lack of uniformity in the finished appearance. The variation in suction characteristics of the surfaces to be painted require corresponding variation of the priming coat, or, in some cases, the use of glue size, petrifying liquid or sealers according to the type of paint to be used. Surfaces which show local variations in suction, as for example, between individual bricks or on patches produced on plastered surfaces by local over-trowelling or by efflorescence, should be treated by the application of a suitable primer.

If the suction is so high or variable that normal painting procedure is likely to give a good finish, one of the following pretreatments should be applied over the whole surface as a primer, according to the type of paint to be used :

Type of paint	*Pretreatment*
(*a*) *Size bound distemper*	A coat of clearcole
(*i*) One coat application	A coat of size alone will be sufficient.
(*ii*) Two-coat application	A coat of the same distemper thinned with water or petrifying liquid Or A coat of 'sharp colour' or primer sealer with the addition of finely ground pumice.
(*b*) *Dry distemper*	
(*c*) *Oil paint*	A coat of thin primer or primer-sealer, preferably in consultation with the manufactures of the paint.
(*d*) *Emulsion paint*	A coat of the same paint thinned with water or sealers recommended by the manufacturer.
(*e*) *Cement paint and lime wash*	Wet the surface before applying paint.

In the case of *new lime plaster* the following points are note-worthy

1. If possible, lime plaster should be left unpainted for the first few months so as to allow the plaster to carbonate, harden and dry thoroughly. If the plaster has any tendency to craze or crack owing to shrinkage on drying, the movements should be allowed to occur before the surface is painted, so as to enable provision of suitable preparatory treatment. Heating the rooms, if accompanied by good ventilation, will assist drying, but should be cautiously adopted.

Too rapid drying may damage the plaster by causing undue shrinkage and separation of the plaster coats.

2. If there is any objection to leaving the plaster base, a temporary decoration of soft distemper (non-washable distemper) may be applied. This may be removed easily at a later date and replaced by a more permanent decoration. Other types of paint suitable for early application are cement paints, silicate paints and washable distemper depending upon the final decoration in view.

3. If the background of the plaster is one likely to contain large amounts of water, for example, new brick work, concrete or building blocks, no attempt should be made to apply oil paint(specially gloss finishes) until there is every reason to believe that the walls are thoroughly dry.

4. If the background is of a dry type, for example, wood or metal lath, oil paints may be applied with the safety after a few weeks drying, and oil-bound distempers even earlier.

20.6. DEFECTS IN PAINTING

The following defects may occur in painting work :

1. ***Blistering.*** It is the defect caused due to the formation of bubbles under the film of paint. The bubbles are formed by water vapours trapped behind the painted surface.

2. ***Bloom.*** In this defect, dull patches are formed on finished polished surface. This may be either due to defect in paint or due to bad ventilation.

3. ***Crawling or sagging.*** This defect occurs due to the application of too thick a paint.

4. ***Fading.*** This is the gradual loss of colour of paint, due to the effect of sunlight on pigments of the paint.

5. ***Flaking.*** Flaking is the dislocation or loosening of some portion of the painted surface, resulting from poor adhesion.

6. ***Flashing.*** It is the formation of glossy patches on the painted surface, resulting from bad workmanship, cheap paint or weather action.

7. ***Grinning.*** This defect is caused when the final coat does not have sufficient opacity so that background is clearly seen.

8. ***Running.*** This defect occurs when the surface to be painted is too smooth. Due to this, the paint runs back and leaves small areas of the surface uncovered.

9. ***Sponification.*** This is the formation of soap patches on the painted surface due to chemical action of alkalies.

20.7. VARNISHING

Varnish is a solution of resins or resinous substances (such as common resin, amber, copal, shellac etc.) in alcohol, turpentine

or oil. It is applied on wood surfaces with the following objects:

(*i*) To intensify or brighten the appearance of natural grains in wood.

(*ii*) To render brilliancy to the painted surface.

(*iii*) To protect painted surface from atmospheric action.

(*iv*) To protect unpainted wooden surfaces of doors, windows, floors, roof trusses etc. from atmospheric action.

Characteristics of a good varnish

A good varnish should possess the following characteristics

1. It should dry quickly.

2. The protective film obtained on drying should be hard, tough, durable and resistant-to wear.

3. The finished surface should be uniform in nature and pleasing in appearance.

4. It should exhibit a glossy surface.

5. It should not shrink or show cracks on drying. It should have sufficient elasticity.

6. The colour of Varnish should not fade a way with time.

Ingradients of varnish

A varnish has the following essential ingradients :

(*i*) Resins or resinous substances.

(*ii*) Solvents.

(*iii*) Driers.

1. *Resins or resinous substances*

The quality of varnish depends largely on the type of resin used. Various types of resins in use are copal, lac or shellac, resin, amber,mastic, gum dammer etc. Copal is a hard and lustrous resin obtained from ground where pine tree existed in past. Resin is obtained from pine trees. Lac or shellac is obtained by exudation of some insects which grow on some type of trees in India. Raw copal, an inferior type, is obtained from standing pine trees :

2. *Solvents*

Different types of solvents are available, but each is used only in conjunction with some specific resin. The following table gives the solvents for different resins :

Type of solvent	*Type of resin*
1. Boiled linseed oil	Amber, copal
2. Methylated sprit of wine	Lac or shellac
3. Turpentine	Mastic, gum dammer, rosin
4. Wood naphtha	Raw copal and other cheap varieties of resin.

3. *Driers*

Driers accelerate the process of drying of a varnish. Common

driers used in varnishes are : litharge, white copper and lead acetate.

Type of varnishes

Varnishes may be divided into the following four categories, depending upon the type of solvent used :

1. Oil varnishes.
2. Spirit varnishes.
3. Turpentine varnishes.
4. Water varnishes.

1. *Oil varnishes*

These varnishes use linseed oil as solvent in which hard resins such as amber and copal are dissolved by heating. These varnishes dry slowly, but form hard and durable surface. Sometimes, small quantity of turpentine is added to make the varnish more workable. Oil varnishes are recommended for all external wood work, and for joinery and fittings.

2. *Spirit varnishes or lacquers*

These varnishes are methylated spirit of wine as solvent in which soft resins such as lac or shellac are dissolved. They dry quickly, but are not durable. French polish is a variety of this type of varnish. It is commonly used on furniture.

3. *Turpentine varnishes*

These varnishes use turpentine as solvent in which soft resins such as gum dammer, mastic and rosin are dissolved. The varnish dries quickly, but is not so durable. These are cheaper than oil varnishes.

4. *Water varnishes*

These varnishes are formed by dissolving shellac in hot water, using enough quantity of either ammonia, borax, potash or soda. Water varnishes are used for varnishing wall papers, maps, pictures, book jackets, etc.

Process of varnishing

Application of varnish on wood work is carried out in the following steps :

1. ***Preparation of surface.*** The wood surface is made smooth by thoroughly rubbing it by means of sand paper or pumice stone.

2. ***Knotting.*** The process of knotting is carried out exactly in the same way as adopted for painting wood work.

3. ***Stopping.*** Stopping is done by means of hot weak *glue size* so that pores on the surface are filled up. Alternately, boiled linseed oil can be applied in two coats. The dry surface should then be rubbed down with sand paper.

4. ***Coat of varnish.*** On the cleaned surface, two or more coats of varnish are applied. Next coat is applied only when the previous coat has dried up thoroughly.

20.8. DISTEMPERING

Distempers are considered to be *water-paints.* A distemper is composed of the following :

1. A base, such as whiting or chalk.
2. A carrier (water).
3. A binder, such as glue or casein.
4. Colouring pigments.

Water-bound distempers are available in powder or paste form, and thcy are mixed with hot water before use. *Oil bound distempers* are a variety of oil paint, in which the drying oil is so treated that it mixes with water. Glue or casein is the emulsifying agent. Oil bound distempers are washable. Distempers are cheaper than oil paints. They are generally light in colour and they provide good reflective coating. However, they are less durable than oil paints.

Process of distempering

Distempering is carried out in the following steps :

1 ***Preparation of surface***

The surface to be distempered should be thoroughly rubbed and cleaned. The efflorescence patches should be carefully wiped out by clean cloth. The irregularities in surfaces (such as cracks, holes, etc.) should be filled with putty. If distempering is to be done on new surface, it should be kept exposed for 3 to 6 months so that all the moisture evaporates. If distempering is to be done on old surface, old loose distemper should be removed by scraping, and profuse watering. New lime plastered surface should be washed with the solution of 1 : 50 sulphuric acid, left for 24 hours and then washed again with clean water. New cement plastered surface should be washed with solution containing 1 kg of zinc sulphate on 10 litres of water, and then allowed to dry.

2. ***Priming coat***

After cleaning the prepared surface, priming coat should be applied. For readymade distempers, priming coat as suggested by the manufacturers should be applied. For locally prepared distempers, milk is used for priming coat. One litre of milk covers about 10 square metre of surface.

3 ***Coats of distemper***

Distemper is applied in 2 to 3 coats. However, next coat should be applied only when the previous coat has dried up and become hard. Distempering should preferably be done in dry weather, to achieve best results.

20.9 WHITE WASHING AND COLOUR WASHING

White washing and colour washing of surfaces of building is neeessary on both hygienic and aesthetic reasons. In order to obtain a clean, neat and uniform finish, it is necessary to adopt proper

method for both preparation of surface to receive white wash or colour wash and for application of white wash or colour wash.

Preparation of white wash

White wash is prepared from fat lime. The lime is slaked at the site and mixed and stirred with about five litres of water for 1 kg of unslaked lime to make a thin cream. This should be allowed to stand for a period of 24 hours, and then should be screened through a clean coarse cloth. One kg of gum is dissolved in hot water to each m^3 of lime cream. About 1.3 kg of sodium chloride dissolved in hot water may also be added for every 10 kg of lime. Sometimes, rice is used in the place of gum. The application of sodium chloride (common salt) to lime-wash helps in quick carbonation of calcium hydroxide making the coating hard and rub-resistant. Small quantity of ultra-marine blue (upto 3 g per kg of lime) may be added to the last two coats of white wash solution.

Preparation of surface

The new surface should be thoroughly cleaned off all dirt, dust mortar drops and other foreign matter before white wash is to be applied. Old surfaces already white-washed or colour-washed should be broomed to remove all dust and dirt. All loose scales of lime wash and other foreign matter should be removed. Where heavy scaling has taken place, the entire surface should be scraped clean. Any growth of moulds moss should be removed by scraping with steel scraper and ammonical copper solution consisting of 15 g of copper carbonate dissolved in 60 ml of liquor ammonia in 500 ml water, should be applied to the surface and allowed to dry thoroughly before applying white or colour wash.

Application of white wash

White wash is applied with *moonj* or other brush, to the specified number of coats(generally three). The operation in each coat should consist of a stroke of the brush given from the top down-wards, another from the bottom upwards over the first stroke, and similarly a stroke horizontally from the right and another from the left before it dries. Each coat should be allowed to dry before the next coat is applied. The white washing on ceiling should be done prior to that on walls.

Colour washing

Colour washing is prepared by adding colouring pigment to the screened white wash. Generally used pigments are yellow earth red ocher and blue vitroil. These are crushed to powder, before mixing. The colour wash is applied in the same fashion as the white wash. For colour washing on new surface, the first primary coat should be of white wash and the subsequent coats (min. two) should be of colour wash.

PROBLEMS

1. Explain in brief the characteristics of (*a*) good paint, (*b*) good varnish.

2. Describe the constituents of a paint, mentioning the specific functions of each.

3. Describe the various types of paints, and their suitability or use.

4. Explain the procedure of painting :
   (*a*) Wood surfaces
   (*b*) Plastered surfaces
   (*c*) Iron and steel surfaces

5. Explain various defects in painting.

6. Describe the constituents of a varnish.

7. Describe various types of varnishes.

8. Write a note on 'distempers' and 'distempering'.

9. Write a note on white washin$_g$ and colour washing.

21

Damp Proofing

21.1. INTRODUCTION : CAUSES OF DAMPNESS

One of the basic requirement of a building is that it should remain dry or free from moisture travelling through walls, roofs or floors. *Dampness* is the presence of hygroscopic or gravitational moisture. Dampness gives rise to unhygienic conditions, apart from reduction in strength of structural components of the building. Damp prevention is therefore one of the important items of building design. Every building should be *damp proof*. Provision of *damp proof courses* prevent the entry of moisture in the building.

Following are various causes of dampness in buildings :

1. *Moisture rising up the walls from ground*

All the structures are founded on soils, and the sub-structure is embedded into it. If the soil is pervious, moisture constantly travels through it. Even in the case if impervious soils, lot of soil moisture may be present. This moisture may rise up into the wall and the floor through capillary action. Ground water rise will also result in moisture entry into the building through walls and floor.

2. *Rain travel from wall tops*

If the wall tops are not properly protected from rain penetration, rain will enter the wall and will travel down. Leaking roofs will also permit water to enter.

3. *Rain beating against external walls*

Heavy showers of rain may beat against the external faces of walls and if the walls are not properly treated, moisture will enter the wall, causing dampness in the interior. If balconies and chajja projections do not have proper outward slope, water will accumulate on these and could ultimately enter the walls through their junction. This moisture travel would completely deface interior decoration of the wall.

4. *Condensation*

Due to condensation of atmospheric moisture, water is deposited on the walls, floors and ceilings. This moisture may cause dampness.

5. *Miscellaneous causes*

Moisture may also enter due to the following miscellaneous causes :

(*i*) *Poor drainage* at the building site.

(*ii*) *Imperfect orientation* : Walls getting less sunlight and heavy showers may remain damp.

(*iii*) *Imperfect roof slope* : Specially in the case of flat roofs.

(*iv*) *Defective construction* : Imperfect wall jointings, joints in roofs, defective throating etc.

(*v*) *Absorption of water* from defective rain water pipes.

21.2. EFFECTS OF DAMPNESS

The following are the ill effects of entry of dampness :

1. Dampness gives rise to breeding of mosquitoes and create unhealthy living conditions.
2. Travel of moisture through walls and ceiling may cause unsighty patches.
3. Moisture travel may cause softening and crumbling of plaster, specially lime plaster.
4. The wall decoration (*i.e.*. painting etc.) is damaged, which is very difficult and costly to repair.
5. Continuous presence of moisture in the walls may cause efflorescence resulting in disintegration of bricks, stones, tiles, etc., and consequent reduction in strength.
6. The flooring gets loosened because of reduction in the adhesion when moisture enters through the floor.
7. Timber fittings, such as doors, windows, almirahs, wardrobes etc., coming in contact with damp walls, damp floors etc., get deteriorated because of warping, buckling, dry-rotting etc. of timber.
8. Electrical fittings get deteriorated, giving rise to leakage of electricity and consequent danger of short circuiting.
9. Floor coverings are damaged. On damp floors, one can not use floor coverings.
10. Dampness promotes and accelerates growth of termites.
11. Dampness along with warmth and darkness breeds germs of dangerous diseases such as tuberculosis, neuralgia,

rheumatism etc. Occupants may even be asthamatic.

12. Moisture causes rusting and corrosion of metal fittings attached to walls, floors and ceilings.

21.3. METHODS OF DAMP PROOFING

Following methods are adopted to make a building damp proof :

(1) Use of damp proofing course (D.P.C.) : membrane damp proofing.
(2) Integral damp proofing.
(3) Surface treatment.
(4) Cavity wall construction.
(5) Guniting.
(6) Pressure grouting.

1. Membrane damp proofing : Use of D.P.C.

This consists of introducing a water repellent membrane or *damp proof course* (D.P.C.) between the source of dampness and the part of building adjacent to it. Damp proofing course may consist of flexible materials such as bitumen, mastic asphalt, bituminous felts, plastic or polythene sheets, metal sheets, cement concrete etc. Damp proofing course may be provided either horizontally or vertically in floors, walls etc. The following *general principles* should be kept in mind while providing D.P.C. :

(*i*) The damp proofing course should cover the full thickness of walls, excluding rendering.

(*ii*) The mortar bed supporting D.P.C. should be levelled and even, and should be free from projections, so that D.P.C. is not damaged.

(*iii*) D.P.C. should be so laid that of a continuous projection is provided.

(*iv*) At junctions and corners of walls, the horizontal D.P.C. should be laid continuous.

(*v*) When a horizontal D.P.C. (*i.e.*. that of a floor) is continued to a vertical face, a cement concrete fillet of 7.5 cm radius should be provided at the junction.

(*vi*) D.P.C. should not be kept exposed on the wall surface otherwise it may get damaged during finishing work.

2. Integral damp proofing

This consists of adding certain water proofing compounds of materials to the concrete mix, so that it becomes impermeable. These water proofing compounds may be in three forms :

(*i*) Compounds made from chalk, talc, fullers earth, which

may fill the voids of concrete under the mechanical action principle.

(*ii*) Compounds like alkaline silicates, aluminium sulphate, calcium chlorides, etc. which react chemically with concrete to produce water proof concrete.

(*iii*) Compounds, like soap, petroleum, oils, fatty acid compounds such as stearates of calcium, sodium, ammonia etc. work on water repulsion principle. When these are mixed with concrete, the concrete becomes water repellent.

(*iv*) Commercially available compounds like Publo, Permo, Silka etc.

3. Surface treatment

The surface treatment consists of application of layer of water repellent substances or compounds on these surfaces through which moisture enters. The use of water repellent metallic soaps such as calcium and aluminium oletes and stearates are much effective against rain water penetration. Pointing and plastering of the exposed surfaces must be done carefully, using water proofing agents like sodium or potassium silicates, aluminium or zinc sulphates, barium hydroxide and magnesium sulphates etc. It should be noted that surface treatment is effective only when the moisture is superficial and is not under pressure. Sometimes, exposed stone or brick wall face may be sprayed with water repellent solutions.

4. Cavity wall construction

This is an effective method of damp prevention, in which the main wall of a building is shielded by an outer skin wall, leaving a cavity between the two. For details about cavity wall construction, reference may be made to chapter 9.

5. Guniting

This consists of depositing under pressure, an impervious layer of rich cement mortar over the exposed surfaces for water proofing or over pipes, cisterns etc. for resisting the water pressure. Cement mortar consists of 1 : 3 cement sand mix, which is shot on the cleaned surface with the help of a cement gun, under a pressure of 2 to 3 kg/cm^2. The nozzle of the machine is kept at a distance about 75 to 90 cm from the surface to be gunited. The mortar mix of desired consistency and thickness can be deposited to get an impervious layer. The layer should be properly cured atleast for 10 days.

6. Pressure grouting

This consists of forcing cement grout, under pressure, into cracks, voids, fissures etc. present in the structural components of

the building, or in the ground. Thus the structural components and the foundations which are liable to moisture penetration are consolidated and are thus made water-penetration-resistant. This method is quite effective in checking the seepage of raised ground water through foundations and sub-structure of a building.

21.4. MATERIALS USED FOR DAMP PROOFING COURSE

An ideal damp proofing material should have the following *characteristics*:

(1) The material should be perfectly impervious and it should not permit any moisture penetration or travel through it.
(2) The material should be durable, and should have the same life as that of the building.
(3) The material should be strong, capable of resisting super-imposed loads/pressure on it.
(4) Material should be flexible, so that it can accommodate the structural movements without any fracture.
(5) The material should not be costly.
(6) The material should be such that leak-proof jointing is possible.
(7) The material should remain steady in its position when once applied. It should not allow any movement in itself.

Following materials are commonly used for damp-proofing course :

1. Hot bitumen

This is highly flexible material, which can be applied with a minimum thickness of 3 mm. It is placed on the bedding of concrete or mortar, while in hot condition.

2. Mastic asphalt

Mastic asphalt is semi-rigid material which is quite durable and completely impervious. It is obtained by heating asphalt with sand and mineral fillers. However, it should be laid very carefully, by experienced persons. It can withstand only very slight distortion. It is also liable to squeeze out in very hot climate or under heavy pressure.

3. Bituminous or asphaltic felts

This is a flexible material which is available in rolls of various wall thicknesses. It is laid on a levelled flat layer of cement mortar. An overlap of 10 cm is provided at joints and full width overlap is provided at angles, junctions and crossings. The laps should be sealed with bitumen. Bituminous felts cannot withstand heavy loads, through they can accommodate slight movements.

4. Metal sheets

Sheets of lead, copper aluminium can be used as D.P.C. These sheets are of flexible type. *Lead sheets* are quite flexible. Their thickness should be such that its weight is not less than 20 kg/m^2 They are laid similar to the bituminous felts. Lead sheets have the advantages of being completely impervious to moisture, resistant to ordinary atmospheric corrosion, capability of taking complex shapes without fracture and resistant to sliding action. It does not squeeze out under ordinary pressure. However, it may be corroded when in contact with lime or cement. It should, therefore, be protected by a coating of bitumen. *Copper sheets*, of minimum 3 mm thickness, are embedded in lime or cement mortar It has high durability, high resistance to dampness, high resistance to sliding and reasonable resistance to ordinary pressure. *Aluminium sheets*, if used, should be protected with a layer of bitumen. It is not as good as lead or copper sheets.

5. Combination of sheets and bituminous felts

Lead foil sand wiched between asphaltic or bituminous felts can be effectively used as D.P.C. The combination, known as *lead core* possesses characteristics of easy laying, durability, efficiency, economy and resistance to cracking.

6. Bricks

Special bricks, having water absorption not less than $4\frac{1}{2}$ % of their weight may be used as D.P.C. in locations where damp is not excessive. These bricks are laid in two to four courses in cement mortar. The joints of bricks are kept open.

7. Stones

Dense and sound stones, such as granite, trap, slates, etc. are laid in cement mortar (1:3) in two courses or layers to form effective D.P.C. The stones should extend to the full width of the wall.

8. Mortar

Cement mortar (1 : 3) is used as bedding layer for housing other D.P.C. materials. A small quantity of lime may be added to increase workability of the mortar. In water used for mixing, 75 gm of soft soap is dissolved per litre of water. This mortar may also be used for plaster work on external walls.

9. Cement concrete

Cement concrete of 1 : 2 : 4 mix or 1 : $1\frac{1}{2}$: 3 mix is generally provided at plinth level to work as D.P.C. The thickness may vary

from 4 cm to 15 cm. Such a layer can effectively check the water rise due to capillary action. Where dampness is more, two coats of hot bitumen paint may be applied on it.

10. Plastic sheets

This is relatively a new type D.P.C. material, made of black polythene, 0.5 to 1 mm thick in the usual walling width and roll lengths of 30 m. C.B.R.I. Roorkee has recently suggested a new D.P.C. which comprises a 400 gauge thick alkathene laid over 12 mm thick 1 : 4 cement mortar. The treatment is cheaper but is not permanent.

21.5. D.P.C. TREATMENT IN BUILDINGS

1. Treatment to foundations against gravitational water

Foundation may receive water percolating from adjacent ground, and this moisture may rise in the wall. This can be checked by providing *air drain* parallel to the external wall. The width of air drain may be about 20 to 30 cm. The outer wall of the drain is kept above the ground to check the entry of surface water. A R.C.C. roof slab is provided. Openings with gratings are provided at regular interval, for the passage of air. Usual D.P.C. are also provided horizontally and vertically, as shown in Fig. 21.1.

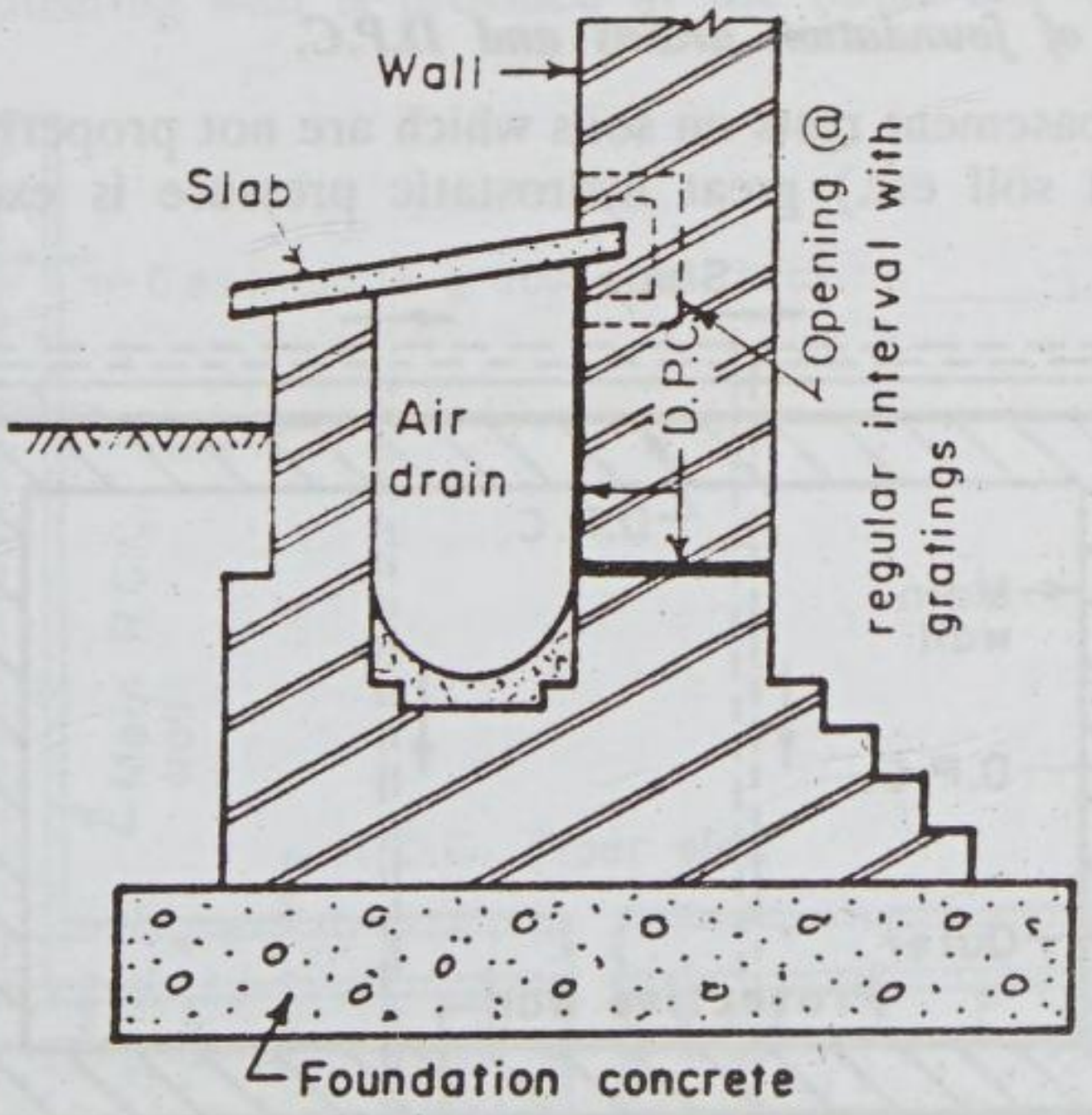


FIG. 21.1. AIR DRAIN.

2. Treatment to basements

When basements in damp soils are constructed, three methods may be adopted :

(*i*) Provision of foundation drains and D.P.C.

(*ii*) Provision of R.C.C. raft and wall slab.
(*iii*) Asphalt tanking.

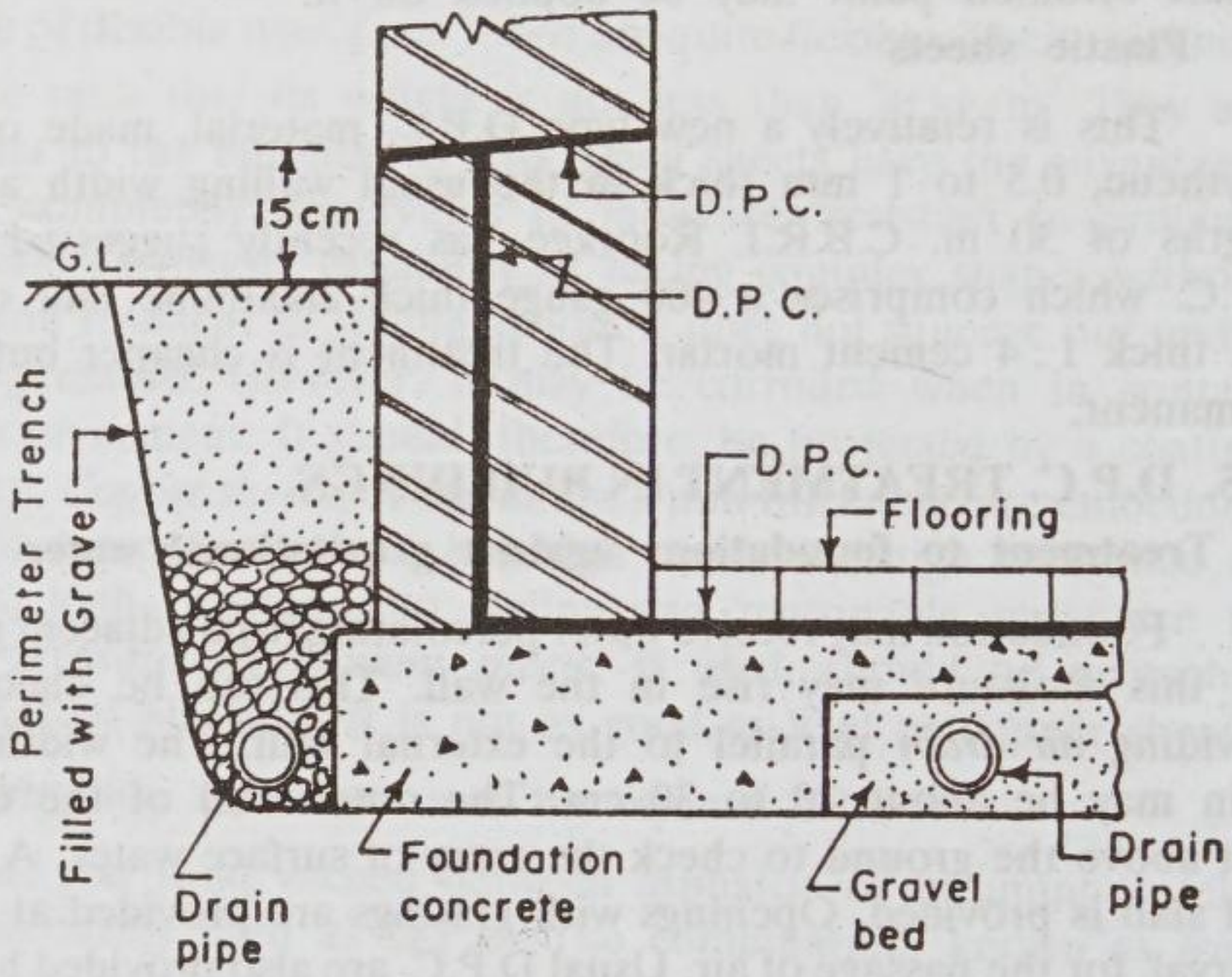


FIG. 21.2. D.P.C. TREATMENT FOR BASEMENT ON UNDRAINED SOILS.

(a) Provision of foundation drains and D.P.C.

When basement rests on soils which are not properly drained, (such as peat soil etc.) great hydrostatic pressure is exerted and

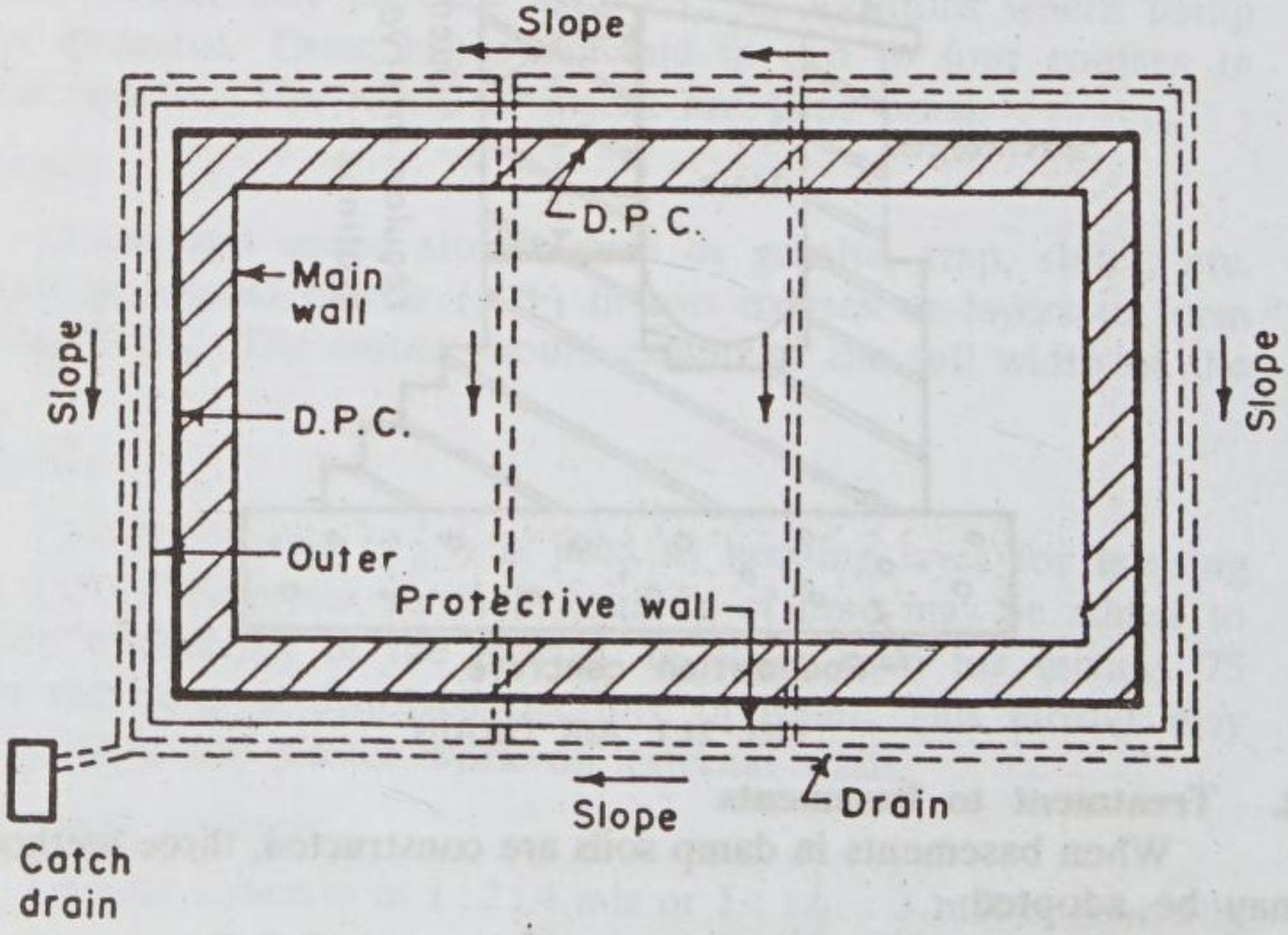


FIG. 21.3. PLAN SHOWING LAYOUT OF DRAINS.

the floor as well as wall receive water continuously oozing out. In such a case it becomes necessary to make a trench all round, upto foundation level and fill it with gravel, coke and other pervious materials. Open jointed drains may be provided to collect the underground water. Drainage pipes, embedded in gravel bed, may also be provided before foundation concrete, as shown in Fig. 21.2. Horizontal and vertical D.P.C. are provided in wall as well as foundation concrete.

The drain may have suitable longitudinal slope, ultimately draining the water into a catch drain. Drain pipes under the basement slab may be provided at some suitable interval, as shown in Fig. 21.3.

(b) Provision of R.C.C. raft and wall slab

Where underground water pressure is severe, the drainage system may not solve the problem effectively. Also, constant pumping out water may be costly. In such a case, floor slab as well as walls may be constructed in rigid R.C.C. structure. Horizontal and vertical D.P.C. treatment is also provided as shown in Fig. 21.4. Atleast 3 layers of bituminous felts are used as D.P.C. Half- brick thick outer protecting wall is provided at the outer face of R.C.C. wall slab.

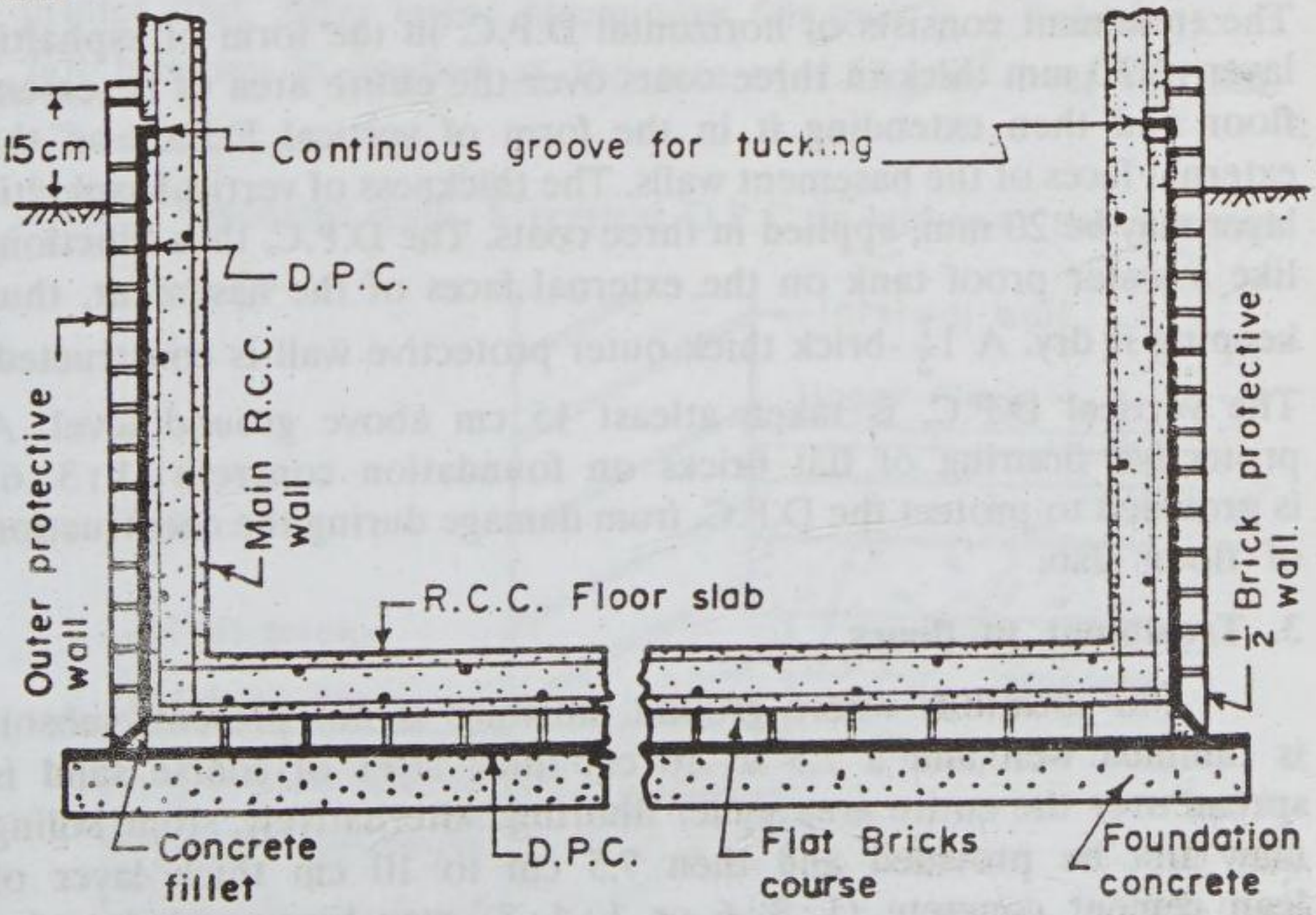


FIG. 21.4. D.P.C. TREATMENT FOR BASEMENT IN DAMP SOIL.

(c) *Asphalt tanking* (Fig. 21.5)

This is adopted when the subsoil water table is not very high.

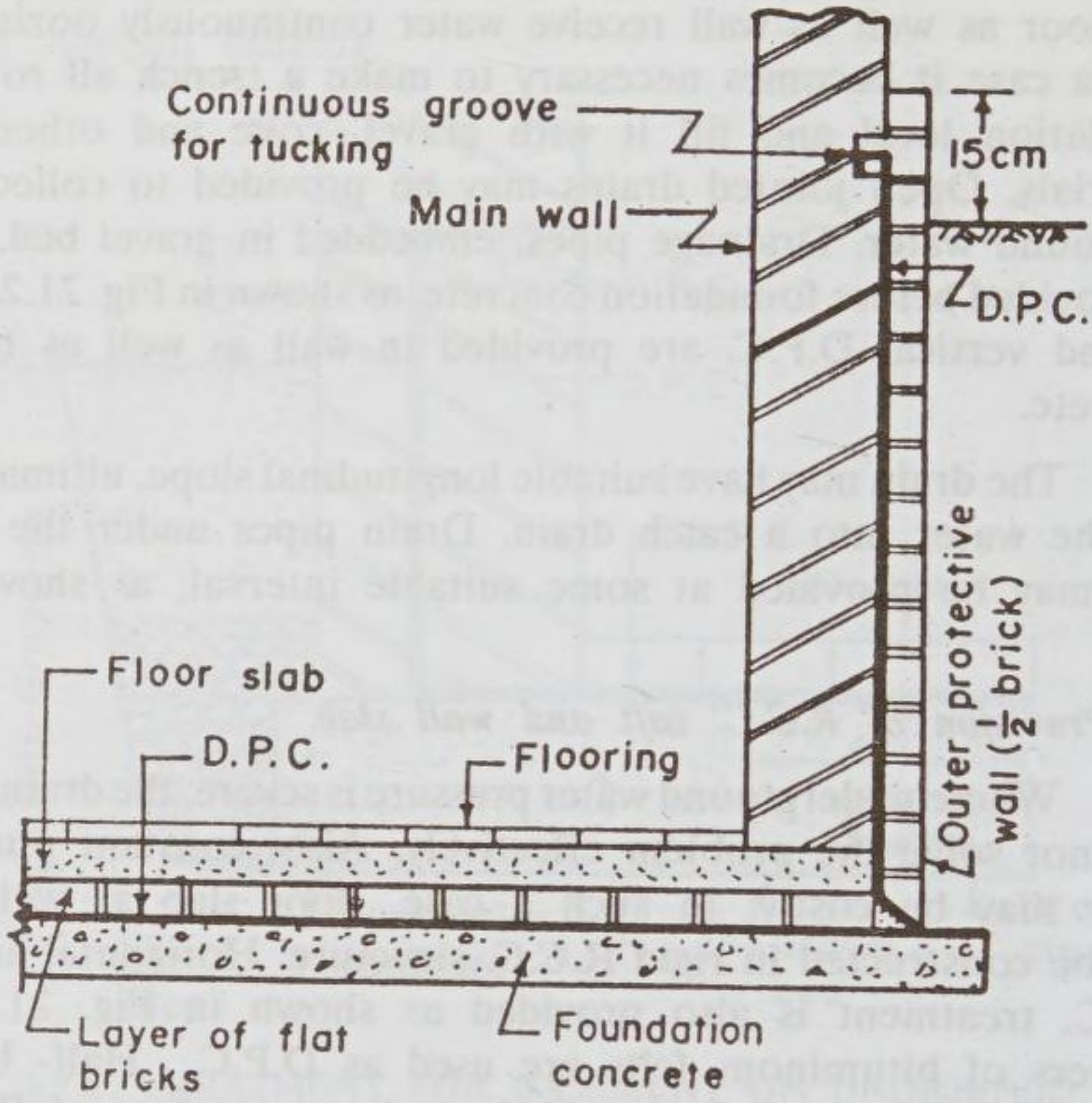


FIG. 21.5. ASPHALT TANKING FOR BASEMENT.

The treatment consists of horizontal D.P.C. in the form of asphaltic layer of 30 mm thick in three coats over the entire area of basement floor and then extending it in the form of vertical D.P.C. on the external faces of the basement walls. The thickness of vertical asphaltic layer may be 20 mm, applied in three coats. The D.P.C. thus functions like a water proof tank on the external faces of the basement, thus keeping it dry. A $1\frac{1}{2}$ -brick thick outer protective wall is constructed. The vertical D.P.C. is taken atleast 15 cm above ground level. A protective flooring of flat bricks on foundation concrete (1 : 3 : 6) is provided to protect the D.P.C. from damage during the construction of floor slab.

3. Treatment to floors

For locations where ground moisture is not present, subsoil is rammed well and a 7.5 to 10 cm thick layer of coarse sand is spread over the entire area under flooring. Alternatively, stone soling may first be provided and then 7.5 cm to 10 cm thick layer of lean cement concrete (1 : 3 : 6 or 1 : 4 : 8) may be provided under it. Over this base, flooring may be laid. However, in damp soils, where water table is near ground surface, it is essential to provide membrane D.P.C. over the entire area, as shown in Fig. 21.6. The membrane may be in the form of mastic asphalt or fibrous asphalt

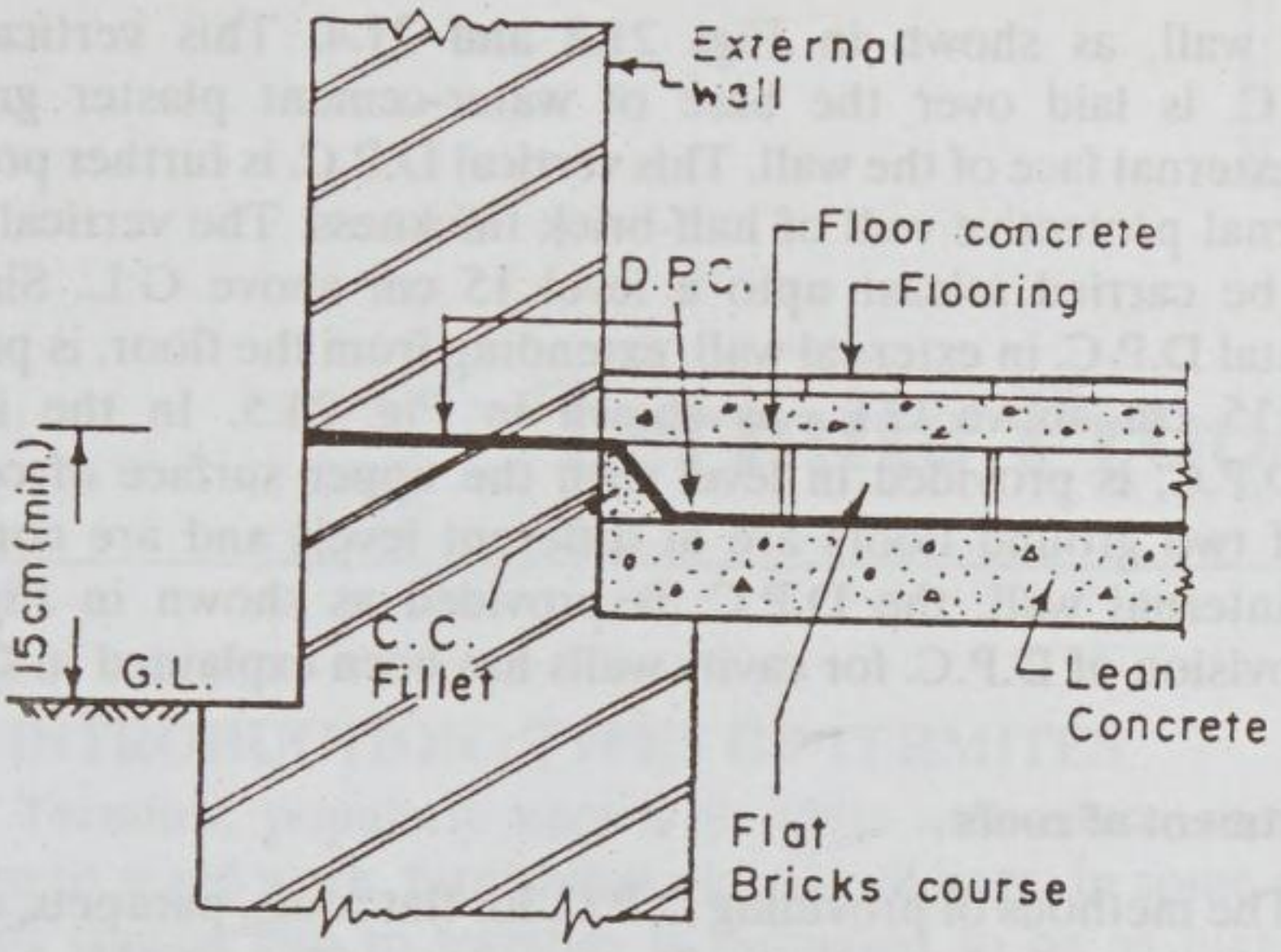


FIG. 21.6. D.P.C. FOR FLOORING.

felt. A layer of flat bricks is laid on a cushion of find sand over D.P.C. to protect it from damage during the construction of floor slab.

Before laying bituminous felt, a coat of hot bitumen, at the rate of 1.5 kg/m^2 is applied over the foundation concrete, to serve as primer coat. After laying bituminous felt over it, a finishing coat of hot bitumen is applied at the rate of 1.5 kg/m^2 over the felt.

4. Treatment to walls

For basement walls, a vertical D.P.C. is laid over the external

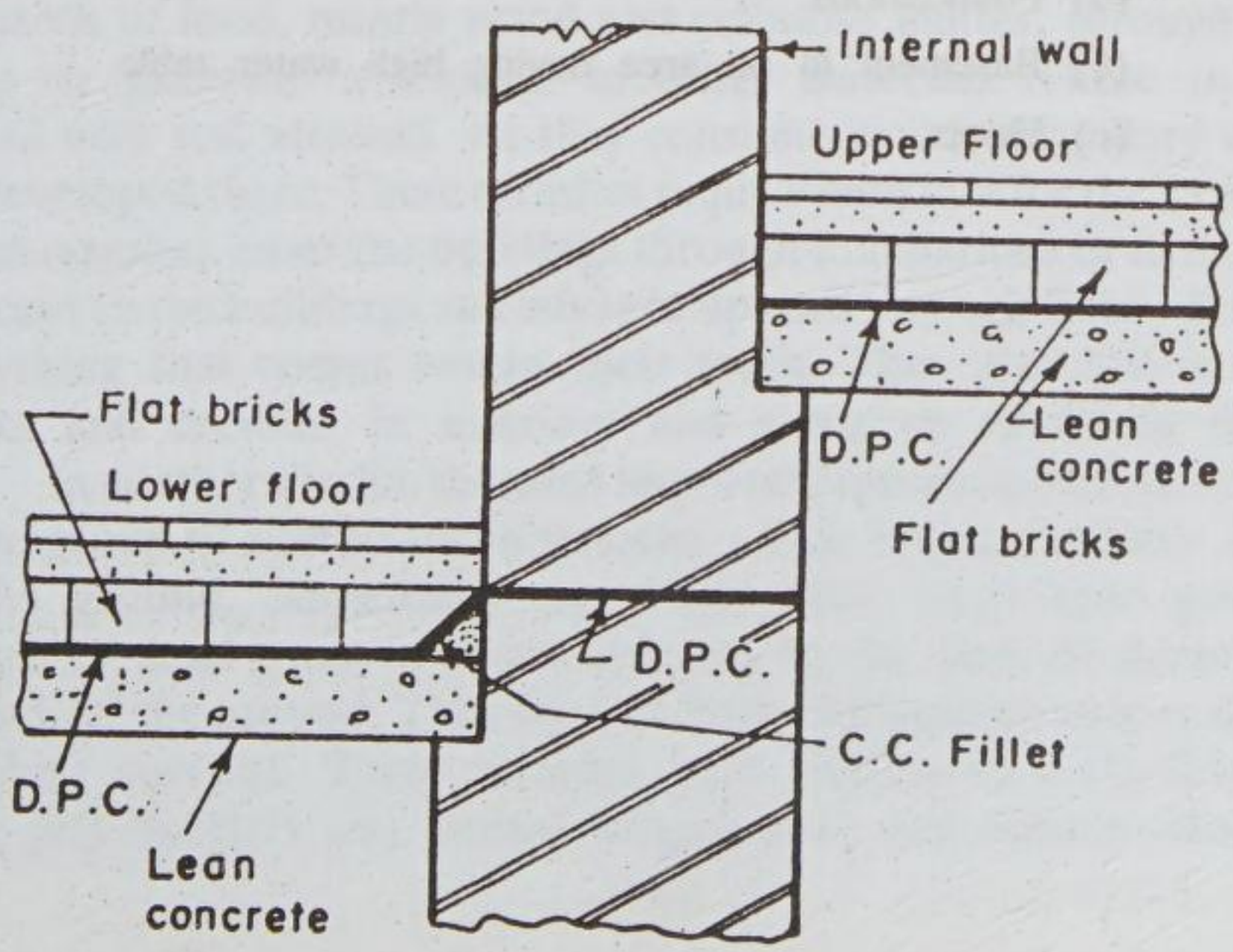


FIG. 21.7. D.P.C. FOR INTERNAL WALL.

face of wall, as shown in Figs 21.3 and 21.4. This vertical layer of D.P.C. is laid over the base of water-cement plaster grounted on the external face of the wall. This vertical D.P.C. is further protected by external protective wall of half-brick thickness. The vertical D.P.C. should be carried atleast upto a level 15 cm above G.L. Similarly, horizontal D.P.C. in external wall, extending from the floor, is provided atleast 15 cm above G.L., as shown in Fig. 21.5. In the internal walls, D.P.C. is provided in level with the upper surface of concrete floor. If two ground floors are at different levels and are connected by an internal wall, the D.P.C. is provided as shown in Fig. 21.6. The provision of D.P.C. for cavity walls has been explained in Chapter 9.

5. Treatment of roofs

The methods of providing D.P.C. for flat roofs, parapets, copings and pitched roofs have been illustrated in Chapter 15.

PROBLEMS

1. (*a*) Explain various causes of dampness in buildings.
   (*b*) What are ill effects of dampness in buildings.
2. Describe various methods of damp proofing.
3. (*a*) Explain various methods used for damp proofing course.
   (*b*) What are the requirements of an ideal material for damp proofing ?
4. Describe the method of damp proofing for the following
   (*a*) Foundations.
   (*b*) Basement in an area having high water table.
   (*c*) Floors.

22

Termite Proofing

22.1. INTRODUCTION : TYPES OF TERMITES

Termites, popularly known as white ants cause considerable damage to wood work, furnishings etc. of buildings. In some countries the loss caused due to termites is estimated to be as high as 10% of the capital outlay of the buildings. *Anti- termite treatment* is, therefore, necessary so that damages are either reduced or stopped all together.

Termites are of two types :

(*i*) Dry wood termites (*ii*) Subterranean termites.

Dry wood termites These termites live in dry wood in small colonies, without maintaining any connection with the soil. They are generally found in humid coastal areas. In India, they are found on coastal regions of South India, though their number is low. They travel and work through wooden structures only.

Subterranean termites These termites have their main colonies in soil, under ground. They cannot survive without maintaining any connection with their prime colonies in the soil. However, they travel in search of food, mostly wood and cellulose matter, through shelter tubes or galleries or tunnels in other materials. These tubes are coated with soil alround. As they consume wood, secondary colonies are developed there. These termites require moisture for their existence. These termites enter the buildings through foundations or from ground adjacent to the buildings and advance upward through floors destroying everything that comes within their reach. They also travel through cracks and crevices in masonry and joints or cracks in floors.

In northern India, the most important species are those belonging to the group of subterranean termites which live in extensive colonies in the ground. Sometimes they build their nests near ground in stumps of dead trees or create colonies in the form of dome-shaped mound on the ground. They require both moisture as well as darkness for their survival. These termites have five *castes* : (*i*) Queen (*ii*) King (*iii*) Soldiers (*iv*) Sexual winged male and female adults, and

(*v*) Workers. Their *workers* forage over extensive areas for edibles, maintaining direct connection with the colony which depends on soil moisture for survival.

A careful examination of untreated building will show that damage by termites and evidence of their activity is not difficult to find. Often such damage or termite activity can be found on the upper floors as well. Even if termite damage on the lower floors is not clearly visible, this should not lead to the erroneous conclusion that they have not established a colony on the upper floors.

22.2. ANTI-TERMITE TREATMENT

Anti-termite treatment may be divided into two categories :

(*i*) Pre-construction treatment.

(*ii*) Post-construction treatment.

(*a*) Pre-construction treatment

This treatment is started right at the initial stage of construction of building. Pre-construction treatment can be divided into three operations :

(*i*) Site preparation.

(*ii*) Soil treatment.

(*iii*) Structural barriers.

22.3. SITE PREPARATION

This operation consists of removal of stumps, roots, logs, waste wood and other fibrous matter from the soil at the construction site. This is essential since the termites thrive on these materials. If termite mounds are detected, these should be destructed by use of insecticide solution, consisting of any one of the following chemicals

Chemical	*Concentration by weight*
(*i*) DDT	5%
(*ii*) BHC	0.5%
(*iii*) Aldrin	0.25%
(*iv*) Heptachlor	0.25%
(v) Chlordane	0.5%

Four litres of the above emulsion in water is required per cubic metre of volume of mound. Holes are made in the mound at several places by use of crow-bar and the insecticide emulsion is poured in these holes.

22.4. SOIL TREATMENT

The best and only reliable method to protect building against termites is to apply a chemical treatment to the soil at the time of construction of the building. This should be done in such a way that a complete chemical barrier is created between the ground from where the termites come and damage wood work in the building. An insecticide solution consists of *any one* of the following chemicals in water emulsion :

Chemical	*Concentration by weight*
(*i*) Aldrin	0.5%
(*ii*) Heptachlor	0.5%
(*iii*) Chlordane	1%

Out of the above chemicals and several other chemicals, *Aldrex* 30 E.C. has proved to be the most effective. It has the following advantages :

(*i*) It is highly toxic to termites.

(*ii*) It can easily be applied after dilution with water.

(*iii*) It is insoluble in water. In other words, this chemical will not dissolve in sub-soil water and disappear quickly from the site.

(*iv*) It is effective even many years after application.

One part of '*Aldrex*' 30 E.C. is diluted with 59 parts of water. This provides an emulsion containing 0.5% *aldrin.*

The emulsion should be applied evenly either with a watering cane or sprayer at the following stages :

Stage 1. In foundation pits, to treat the *bottom* and *sides*

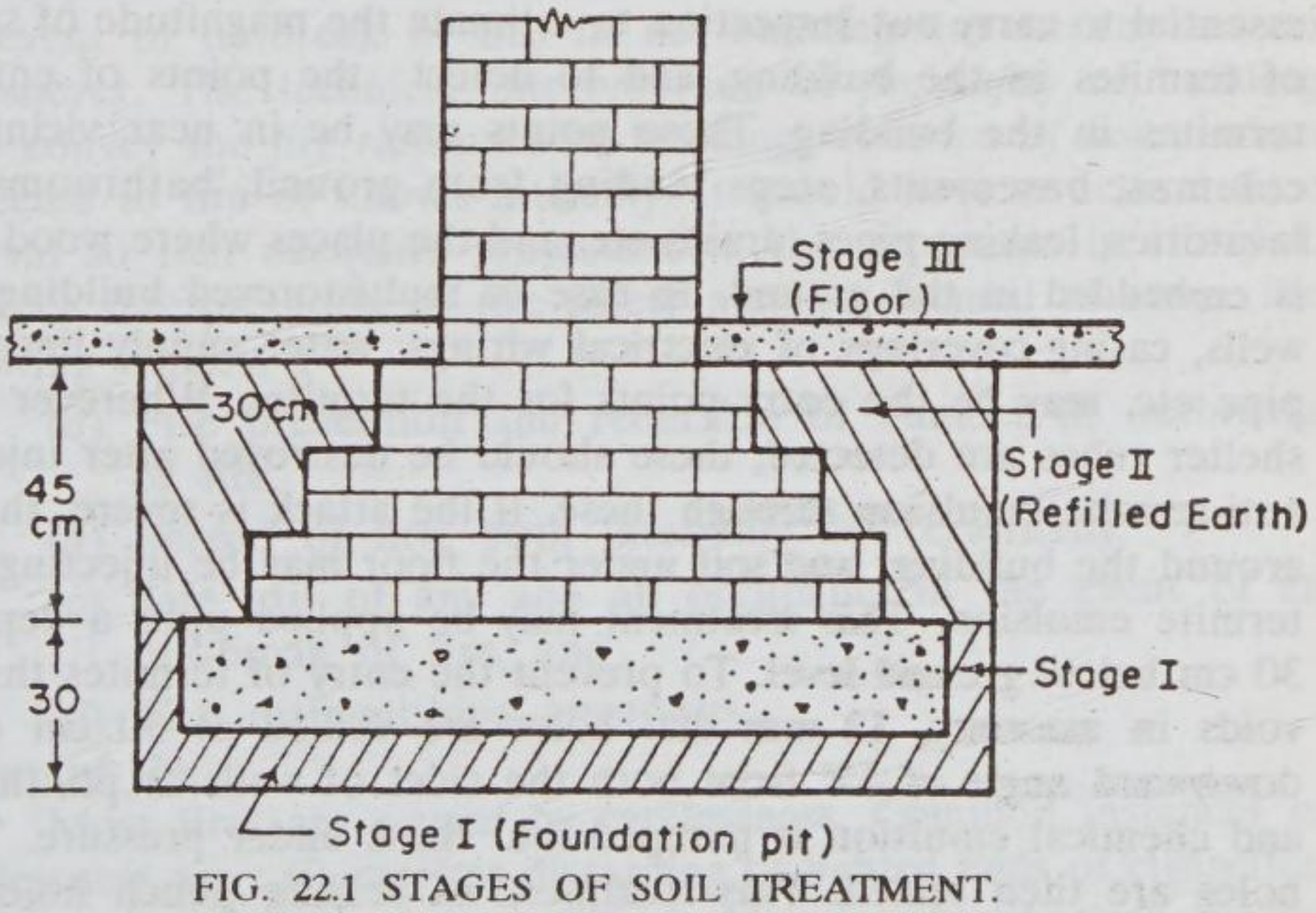


FIG. 22.1 STAGES OF SOIL TREATMENT.

upto a height of about 30 cm. The emulsion required is at the rates of 5 litres per square metre.

Stage 2. The *refill earth* on both the sides of all built up walls, for width of 30 cm and depth of 45 cm approximately. The emulsion required is at the rate of 5 litres per square metre.

Stage 3. Before laying the floor, the entire levelled surface is to be treated at the rate of 5 litres of emulsion per square metre.

The stages of treatment are shown diagrammatically in Fig. 22.1. When used as recommended above, approximately 200 ml of

'Aldrex' 30 E.C. would be required to treat one square metre of the covered area.

22.5. PHYSICAL STRUCTURAL BARRIERS

Continuous impenetrable physical structural barriers may be provided continuously at plinth level to prevent entry to termites through walls. These barriers may be in the form of concrete layer or metal layer. Cement concrete layer may be 5 to 7.5 cm thick. It is preferable to keep the layer projecting about 5 to 7.5 cm internally and externally. Metal barrier may consist of non-corrodible sheets of copper or galvanised iron, of 0.8 mm thick. These sheets are likely to be damaged ; in that case, they become ineffective against termite movement.

22.6. POST-CONSTRUCTION TREATMENT

It is a maintenance treatment for those buildings which are already under attack of termites. As stated earlier, the termites, even after entering the building, maintain their contact with their nest or colony in the ground, through shelter tubes or tunnels lined with soil. This fact is well utilised in the anti-termite treatment. It is essential to carry out inspection to estimate the magnitude of spread of termites in the building, and to detect the points of entry of termites in the building. These points may be in near vicinity of columns, basements, steps leading from ground, bathrooms and lavatories, leaking pipes, drains etc. and the places where wood work is embedded in the ground. In case of multistoreyed buildings, lift wells, casing-coverings of electrical wirings, water supply lines, soil pipe etc. may be the entry points for the termites. Wherever these shelter tubes are detected, these should be destroyed after injecting anti-termite emulsion through these. If the attack is severe, the soil around the building, and soil under the floor may be injecting anti-termite emulsion. This treatment may be applied upto a depth of 30 cm below ground level. To prevent the entry of termites through voids in masonry, 12 mm dia. holes are drilled at 30 cm c/c at downward angle of 45° from both the sides of walls at plinth level and chemical emulsion is pumped into these under pressure. These holes are then sealed. This treatment of drilling punch holes and pumping chemical emulsion is carried out at critical locations such as wall corners, column bases, place of embedment of doors and windows etc. Similar holes are drilled in damaged wood work also and then oil based chemical emulsion is pumped into these.

PROBLEMS

1. Write a note of 'termites and their attack' on buildings.
2. Explain how preconstruction anti-termite treatment is carried out.
3. Explain how post-construction anti-termite treatment is carried out.

23

Fire Protection

23.1. INTRODUCTION

No building material is perfectly fire proof. Every building contains some materials (such as furniture, clothing, eatables etc.) which can either easily catch fire or which are vulnerable to fire. However, the endeavor of the architects and engineers should be to plan, design and construct the building in such a way that safety of occupants may be ensured to the maximum possible extent in the event of outbreak of fire in the building due to any reason whatsoever. The technical interpretation of *fire safety* of building is to convey the *fire resistance* of buildings *in terms of hours* when subjected to fire of known intensity. It should have structural time interval so that adequate protection to the occupants is afforded. A wider interpretation of fire safety may be deemed to cover the following aspects :

(*a*) Fire prevention and reduction of number of outbreaks of fire,

(*b*) Spread of fire, both internally and externally,

(*c*) Safe exit of any and all occupants in the event of an out-break of fire, and

(*d*) Fire extinguishing apparatus.

Causes of fire

Most fires are *caused* by carelessness. Common instances of carelessness are : (*i*) careless discarding of lighted ends of cigarettes, cigars, matches and tobacco, (*ii*) smoking in unauthorised places. (*iii*) indifferent maintenance of machinery including overloading and under or over lubricating of bearings, (*iv*) general indifference to cleanliness, (*v*) incorrect storage of materials, (*vi*) faulty workmanship and inattention to electrical installations (this is particularly evident by the fires which occur during the monsoon), (*vii*) un-approved equipment and layout, (*viii*) inattention of persons concerned with inspection and patrol of the premises under their jurisdiction, and

(*ix*) inattention of fire safety regulations, etc., etc.

In case of an outbreak of fire, the danger is from fire, smoke and panic. The provision of suitable means of escape should be in relation to these dangers and the number of persons affected. The chances of damage due to panic can be reduced ; the *escapes* should be located in such a way that they remain unobstructed by smoke or fumes. The means of escapes from fire should be easily accessible, unobstructed and clearly defined.

23.2 FIRE HAZARDS

Fire safety of buildings should be considered from three aspects and protection should accordingly be provided against the following three types of fire hazards.

(*a*) Possibility of loss or damage to life, referred to as *personal hazard*.

(*b*) Possibility of fire occurring and spreading inside the building itself, referred to as *'internal hazard'* and

(*c*) Possibility of fire spreading from an adjoining building or buildings or from across a street or road, referred to as *'exposure hazard'*.

The consideration of *personal hazard* is naturally of permanent importance and requires the provision of liberally designed and safe fire proof exits and escapes in all building and particularly those having more than one storey.

Internal hazard concerns damage or destruction of the building and influences directly personal hazard. The internal hazard is directly related to *fire load* which, in turn, enables the building to be graded when considered along with the duration of fire..

'Exposure hazard' deals with the risk of fire spreading into a building through the open air from fire in other buildings, from stocks of combustible material etc., or into a division or compartment of a building through the open air from a fire in other division or compartment of the same building.

A small building containing highly inflammable material may constitute a high internal hazard ; a large building containing quantities of combustible material, for example, a godown, would also be described as high internal hazard even though the actual outbreaks are likely to be few, because when a fire does occur, the destruction of contents and structural damage might be considerable. Theatres, cinemas and other places of public assembly, even though their combustible contents may be low, are considered to present a high internal hazard primarily because of the large number of people and the extent of personal hazard, involved. On the other hand, from stand point of high combustible content, would constitute low personal hazard because of few people likely to be in such a building.

23.3. FIRE LOAD

Fire load is the amount of heat in Kilocalories (k cal) which is liberated per square metre of floor area of any compartment by the combustion of the contents of the building and any combustible part of the building itself. This amount of heat is used as the basis of grading of occupancies.

The fire load is determined by multiplying the weight of all combustible materials by their calorific value, and dividing the floor area under consideration. For example, if a section of a building, having an area of 80 sq. metre has 1200 kg of combustible material having a calorific value of 4000 kcal/kg,

$$\text{Fire load} = \frac{1200 \times 4000}{80} = 60000 \text{ kcal/m}^2.$$

Indian Standard (IS : 1641 – 1960) grades the fire loads into the following three *classes* :

(*a*) ***Low fire load*** : not exceeding 275000 kcal/m^2 and as applying generally to domestic buildings, hotels and offices and similar buildings.

(*b*) ***Moderate fire load*** : exceeding 275000 kcal/m^2 but not exceeding 550000 kcal/m^2 applying generally to trading establishment and factories.

(*c*) ***High fire load*** : where the value exceeds 550000 but does not exceed 1100000 kcal/m^2 applying to fire load grading to godowns and similar structures.

Fire load of any building is classed as of normal or of abnormal fire risk depending on susceptibility of the occupancy of the building to fire. The occupancy of the building may consist of materials in store or manufacturing processes.

Different materials having the same weight and the same calorific value may present different hazards on account of their other properties, such as rate of ignition, speed of burning and liberation of dangerous fumes. Materials are classified for purpose of assessing fire grading under the heading Non-Hazardous (NH), Hazardous (H) and Extra Hazardous (EH) based on the following characteristics : (*i*) explosive tendencies, (*ii*) high inflammability, (*iii*) liability to intensify a fire. (*iv*) generation of intense heat when burning, (*v*) liability to extend the fire zone, (*vi*) difficulty to extinguish, and (*viii*) spontaneous combustion tendencies.

Grading of occupancies by fire load

Based on fire load, occupancies are graded into the following three classes :

1. ***Occupancies of low fire load***

Under this fall those occupancies whose the fire load does not exceed an average of 275000 kcal/m^2 of net floor area of any compartment, nor an average of 550000 kcal/m^2 on limited isolated areas. Domestic buildings, hotels, boarding houses, restaurants, schools, hospitals, temples, mosques, commercial offices come under this category. Also, the factories and workshops in which materials and processes are of a recognised non-hazardous nature (such as an engineering workshop) come under this.

2. ***Occupancies of moderate fire load***

Under this fall those occupancies whose the fire load exceeds an average of 275000 kcal/m^2 of net floor area of any compartment but does not exceed an average of 550000 kcal/m^2 nor on average of 1100000 kcal/m^2 on limited isolated areas. Examples of occupancies that fall under this category are retail shops, emporium, bazars, factories and workshops generally.

3. ***Occupancies of high fire load***

Under this fall those occupancies whose fire load exceeds an average of 550000 kcal/m^2 of net floor area of any compartment but does not exceed an average of 1100000 kcal/m^2 of net floor area, nor an average of 2200000 kcal/m^2 on limited isolated areas. Examples of occupancies that fall under this category are godowns and similar buildings used for bulk storage of non-hazardous materials and goods.

23.4. GRADING OF STRUCTURAL ELEMENTS

Structural elements of buildings are graded, for fire resistance, by the time for which they resist a standard fire of given time temperature grading. The time-temperature grading is based on observations in actual fires. The relationship between the actual fire expressed as fire load and the standard fire is established by burning down weights of combustible material corresponding to different classes of fire loads, so as to match the time temperature grading of the standard fire. From the results it follows that the different grades of fire resisting structural elements will resist the corresponding fire loads shown against them in Table 23.1 (IS : 1641-1960).

Thus, a structural element classified as of grade 4 will successfully withstand the standard fire severity and comply with other conditions for an hour. If that structural element is incorporated in a building of which the fire load gives rise to a fire, equivalent in severity

to one hour severity in the test, then the structural element should resist the building fire without failure.

TABLE 23.1.
CLASSIFICATION OF STRUCTURAL ELEMENTS

Grade No	*Time-in-hours (min.resistance against standard fire*	*Fire load and class of fire which the structural element can withstand*	
		Fire load in k cal/m^2	*Class of fire*
1	6	1100000 and over	Very high
2	4	500000 to 1100000	High
3	2	275000 to 500000	Medium
4	1	Less than 275000	Low
5	$\frac{1}{2}$	–	Very low

23.5. GRADING OF BUILDINGS ACCORDING TO FIRE RESISTANCE

Structural precautions aid in giving a building the necessary resistance to a complete burn and restrict any spread of fire and also minimize the personal hazard. In grading building according to fire resistance and structural precautions provided, it has been assumed that no assistance will be forthcoming from municipal fire brigade and that no fire fighting apparatus has been provided or attached to building. National Building Code of India (SP : 7-1970) divides buildings into the following *four* types according to the *fire load* the building is designed to resist :

(*i*) *Type 1 construction.* All structural components have 4-hours fire resistance.

(*ii*) *Type 2 construction.* All structural components have 3-hour fire resistance.

(*iii*) *Type 3 construction.* All structural components have 2-hour fire resistance.

(*iv*) *Type 4 construction.* All structural components have 1-hour resistance.

Experience shows that with fire fighting equipment installed in the premises, the duration of fire in buildings having a fire load between 500000 to 1100000 kcal/m^2 is usually less than 3 hours. Hence type 1 construction prescribed for this class of buildings generally ensures sufficient protection. However, in buildings covered under type 1, proper ventilation and provision for escape of hot gases

should be made. Also, when fire fighting equipment or the services of a fire brigade are available in the premises, the design should provide for immediate access from several positions.

The most satisfactory condition of a building is when it is constructed to resist a complete burn out of combustible contents, without failure or collapse.

23.6. CHARACTERISTICS OF FIRE RESISTING MATERIALS

An ideal fire resisting material should possess the following characteristics :

1. The material should not disintegrate under the effect of great heat.

2. The expansion of the material due to heat should not be such that it leads to instability of the structure of which it forms a part.

3. The contraction of the material due to sudden cooling with water (during fire extinguition process) after it has been heated to a high temperature should not be rapid.

In relation to fire, building materials can be divided into two types : (*i*) *non-combustible materials,* and (*ii*) *combustible materials. Non-combustible materials* are those which if decomposed by heat will do so with absorption of heat (*i.e.* endothermically) or if they oxidise, do so with negligible evolution of heat. These materials do not contribute to the growth or spread of fire, but are damaged and decomposed when high temperatures are reached. Examples of non-combustible materials are : stones and bricks, concrete, clay products, metal, glass etc. *Combustible materials* are those which, during fire, combine exothermically with oxygen, resulting in evolution of lot of heat and giving rise to flame or glow. Such materials burn and also contribute to the growth of fire. Examples of these materials are : wood and wood products, fibre board, straw board etc.

23.7. FIRE-RESISTING PROPERTIES OF COMMON BUILDING MATERIALS

1. Stone

Stone is a non-combustible building material and also a bad conductor of heat and does not contribute to the spread of fire. However, it is a *bad* fire-resisting material since it is liable to disintegrate into small pieces when heated and suddenly cooled, giving rise to failure of structure. Granite, on exposure to severe heat, explodes and disintegrates. Lime stone is the worst, since it is easily crumbled even under ordinary fire. Sand stone of compact composition (fine

grained) can, however, stand the exposure to moderate fire without serious cracks. *In general, the use of stone in a fire-resisting construction should be restricted to a minimum.*

2. Bricks.

Brick is a poor conductor of heat. First class bricks moulded from good clay can stand exposure to fire for a considerable length of time, upto temperatures of about 1200°C. Brick masonry construction, with good mortar and better workmanship, is the most suitable for safeguarding the structure against fire hazards.

3. Concrete

The behavior of concrete during exposure to heat varies with the nature of coarse aggregate and its density, and the quality of cement. It also depends upon the position of steel in concrete. Aggregates expand on heating while ordinary cement shrinks on heating. These two opposite actions may lead to spalling of the concrete surface. Aggregates obtained from igneous rocks containing higher calcareous content, tend to crack more while the aggregates like foamed slag, cinder and bricks are better. The cracks formed in concrete generally extend to a depth of about 25 mm. Hence reinforced concrete fire-resistant construction should have greater cover. *In general, concrete offers a much higher resistance to fire than any other building material.* Reinforced concrete structures can withstand fire lasting for several hours with a temperature of 1000°C without serious damage.

4. Steel

Though steel is non-combustible, it has very low fire resistance, since it is a good conductor of heat. During fire, it gets heated very soon, its modulus of elasticity reduces and it looses its tensile strength rapidly. It is found that yield stress of mild steel at 600° C is about $\frac{1}{2}$ of its value at normal temperatures. Hence unprotected steel beam sags and unprotected columns or struts buckle, resulting in the collapse of structures. If the surface paint on these steel components is not fire resistant structure, it is essential to protect structural steel members with some coverings of insulating materials like brick, terra-cotta, concrete etc. Fixing of steel in plate or sheet form to the structural steel frame work is also effective in resisting the passage of flame. Such construction is widely used in making fire-resisting doors and windows.

5. Glass

Glass is poor conductor of heat, and its thermal expansion is also less. When it is heated and then suddenly cooled, cracks

are formed. These cracks can be minimised if glass is reinforced with steel wire netting. Thus, *reinforced glass* is more fire resistant, and can resist variations in temperature without serious cracks. Reinforced glass has higher melting point. Even if cracks are formed, the embedded wires hold the cracked portion in position. Reinforced glass is therefore commonly used for fire-resisting doors, windows, done sky-lights, etc.

6. Timber

Timber is a combustible material. It ignites and gets rapidly destroyed during fire, if the section is small. However, if timber is used in *thick* sections, it possesses the properties of self-insulation and slow burning. During exposure to fire, timber surface gets charred ; this charred portion acts as protective coating to the inner portion. However, if the temperatures are higher than 500°*C*, timber gets dehydrated under continued exposure, giving rise to combustible volatile gases which readily catch fire. In order to make timber fire-resistant, the following measures are adopted :

(*i*) use of thicker sections at wider spacing than thinner sections at closer spacing, specially in case of floor joints. (*ii*) reducing number of corners and area of exposed surfaces to a minimum. (*iii*) coating timber surface with chemicals like ammonium phosphate and sulphate, borax and boric acid, zinc chloride, (*iv*) painting timber surfaces with asbestos or ferrous oxide paints, if painting is necessary. Painting these with oil paints or varnish should not be done since these paints catch fire.

7. Cast-iron and wrought iron

Cast iron behaves very badly in the event of fire. On sudden cooling, it gets contracted and breaks down into pieces or fragments, giving rise to sudden failure. Hence it is rarely used in fire-resistant building unless suitably covered by bricks, concrete etc. Wrought iron behaves practically in the same way as mild steel.

8. Asbestos cement

It is formed by combining fibrous asbestos with Portland cement. It has low coefficient of expansion and has property of incombustibility. It has, therefore, great fire-resistance. Asbestos cement products are largely used for construction of fire-resistant partition walls, roofs, etc. It is also used as protective covering to other structural members.

9. Aluminium

It is very good conductor of heat. It has very poor fire-resistant properties. Its use should be restricted to only those structures which have very fire risks.

10. Plaster or mortar

Plaster is non-combustible. Hence it should be used to protect walls and ceilings against fire risks. Cement plaster is better than lime plaster since the latter is likely to be calcined during fire. The fire-resistance of plaster can be increased by using it in thick layers or reinforcing it with metal laths. Gypsum plaster, when used over structural steel members, make them better fire-resistant.

23.8. GENERAL FIRE SAFETY REQUIREMENTS FOR BUILDINGS

In order that the fire hazards (i.e. personal hazard, internal hazard and exposure hazards) are minimised, IS : 1641-1960 recommends that the buildings shall conform to the following general requirements :

1. All buildings and particularly buildings having more than one storey shall be provided with liberally designed and safe fire-proof exits or escapes.

2. The exits shall be so placed that they are *always* immediately accessible and each is capable of taking all the persons on that floor as alternative escape routes may be rendered unusable and/or unsafe due to fire.

3. Escape routes shall be well-ventilated as persons using the escapes are likely to be overcome by smoke and/or fumes which may enter from the fire.

4. Fire-proof doors shall conform rigidly to the fire safety requirements.

5. Where fire-resisting doors are employed as cut-offs or fire breaks, they shall be maintained in good working order so that they may be readily opened to allow quick escape of persons trapped in that section of the building, and also, when necessary, prompt rescue work can be expeditiously carried out.

6. Electrical and/or mechanical lifts, while reliable under normal conditions may not always be relied on for escape purposes in the event of a fire, as the electrical supply to the building itself may be cut-off or otherwise interrupted, or those relying on mechanical drive may not have the driving powder available.

7. Lift shafts and stairways invariably serve as flues or tunnels thus increasing the fire by increased drought and their design shall be such as to reduce or avoid this possibility and consequent spread of fire.

8. False ceiling, either for sound effects or air-conditioning

or other similar purpose shall be so constructed as to prevent either total or early collapse in the event of fire so that persons underneath are not fatally trapped before they have the time to reach the exits ; this shall apply to cinemas, and other public or private buildings where many people congregate.

9. To a lesser extent, the provisions of clause (8) above shall apply to single-storey buildings which may be used for residence or an equivalent occupancy. Whatever be the class or purpose of the building, the design and construction shall embody the fire retardent features for ceilings and/or roofs.

10. *Floors.* Floors are required to withstand the effects of fire for the full period stated for the particular grading. The design and construction of floors shall be of such a standard that shall obviate any replacement, partial or otherwise, because experience shows that certain types of construction stand up satisfactorily against collapse and suffer when may first be considered as negligible damage, but in practice later involves complete stripping down and either total or major replacement. This consideration shall also be applied to other elements of structure where necessary.

11. *Roofs.* Roof for the various fire-grades of the buildings shall be designed and constructed to withstand the effect of fire for the maximum period for the particular grading, and this requires concrete or equivalent construction. It is, however, important that maximum endurance is provided for as stated in para 9.

12. *Basements.* Where basements are necessary for a building and where such basements are used for storage, provision shall be made for the escape of any heat arising due to fire and for liberating and smoke which may be caused. It is essential that fire resistance of the basement shall conform to the highest order and all columns for supporting the upper structures shall have a grading not less than laid down in types 1 to 3.

13. *Smoke extraction from basements.* The following requirements shall be provided for smoke extraction :

(*a*) Unobstructed smoke extracts having direct communication with the open air shall be provided in or adjoining the external walls and in positions easily accessible for firemen in an emergency.

(*b*) The area of smoke extracts shall be distributed, as far as possible, around the perimeter to encourage flow of smoke and gases where it is impracticable to provide a few large extracts, for example, not less than $3\,m^2$ in area, a number of small extracts having the same gross area shall be provided.

(*c*) Covers to the smoke extracts shall, where practicable, be provided in the stall board and/or pavement lights at pavement level, and be constructed of light cast iron frame or other construction which may be readily broken by fire-men in emergency. The covers shall be suitably marked.

(*d*) Where they pass through fire resisting separations, smoke extracts shall in all cases be completely separated from other compartments in the building by enclosures of the appropriate grade of fire resistance. In other cases, steel metal ducts may be provided.

(*e*) Where these are sub-basements, the position of the smoke extracts from sub-basements and basements shall be suitably indicated and distinguished on the external faces of the building.

23.9. FIRE RESISTANT CONSTRUCTION

In a fire resistant construction, the design should be such that the components can withstand fire as an integral member of structure, for the desired period. We shall consider the construction of the following components :

1. Walls and columns.
2. Floors and roofs.
3. Wall openings.
4. Escape elements.
5. Strong room construction.

1. Walls and columns

The following points should be observed for making walls and columns fire-resistant :

(*i*) Masonry walls and columns should be made of thicker section so that these can resist fire for a longer time, and can also act as barrier against spread of fore to the adjoining areas.

(*ii*) In the case of solid load-bearing walls, bricks should be preferred to stones.

(*iii*) If walls are to be made of stones, granite and lime stone should be avoided.

(*iv*) In the case of building with framed structure, R.C.C. should be preferred to steel.

(*v*) If steel is used for the framed structure, the steel structural components should be properly enclosed or embedded into concrete, terracotta, brick, gypsum plaster board, or any other suitable material, as illustrated in Fig. 23.1.

(*iv*) If the frame work is of R.C.C., thicker cover should be used so that the members can resist fire for a longer time. It is

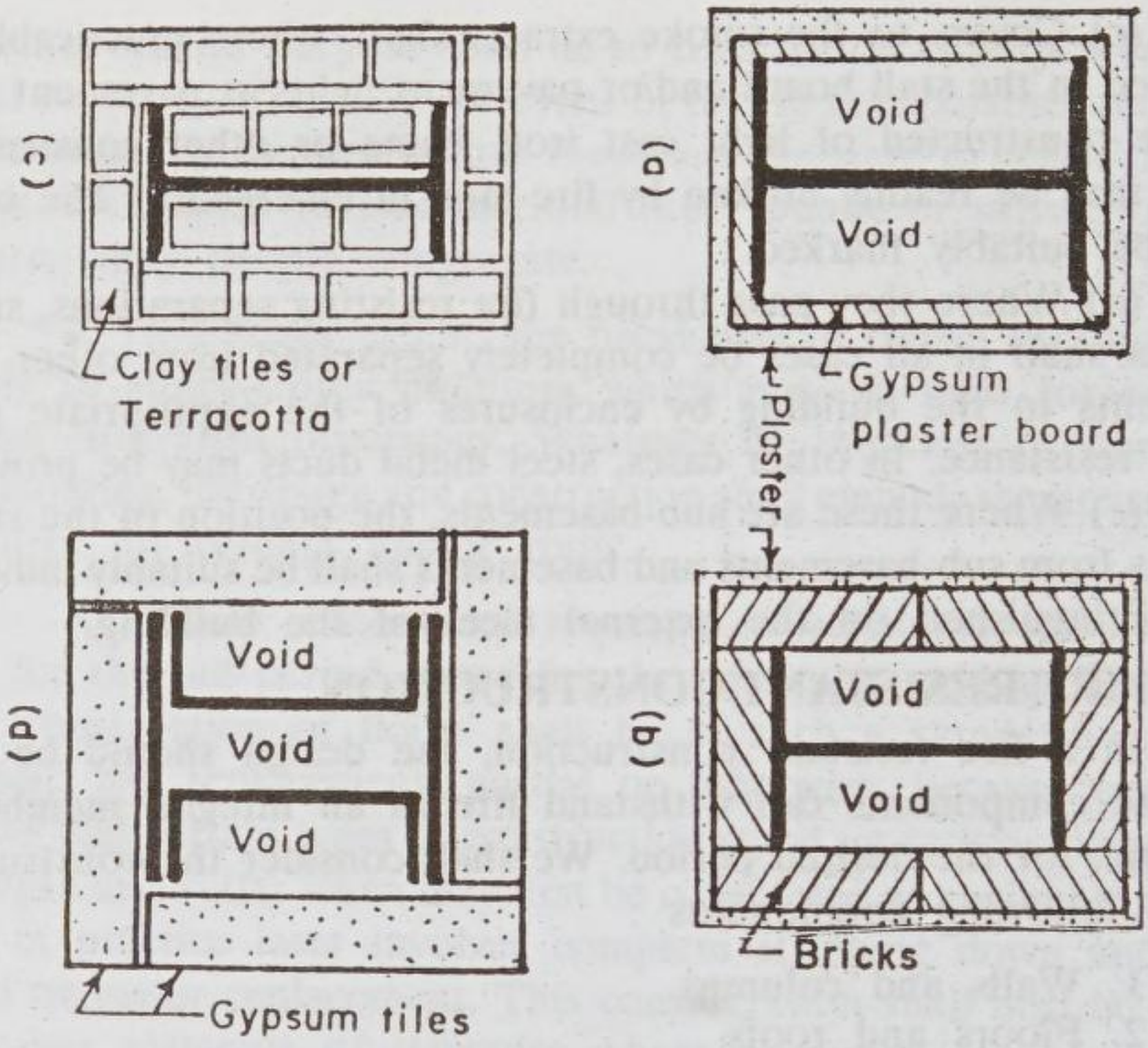


FIG. 23.1. PROTECTION OF STEEL COMPONENTS.

recommended to use 40 to 50 mm cover for columns, 35 to 40 mm cover for beams and long span slabs and 25 mm for short span slabs.

(*vii*) Partition walls should be of fire-resistant materials such as R.C.C., reinforced brick work, hollow concrete blocks, burnt clay tiles, reinforced glass, asbestos cement boards or metal laths covered with cement plaster.

(*viii*) Cavity wall construction has better fire resistance.

(*ix*) All walls, whether load bearing or non-load bearing, should be plastered with fire-resistive mortar.

2. Floors and roofs

The following points are note-worthy for fire-resistant floors and roofs :

(*i*) For better fire resistance, slab roof is preferred to sloping or pitched roofs.

(*ii*) If it is essential to provide sloping roof, trusses should either be of R.C.C. or of protected rigid steel with fire proof covering.

(*iii*) For better fire resistance, the floor should be either of

R.C.C. or of hollow tiled ribbed floor or of concrete jack arch floor with steel joists embedded in concrete.

(*iv*) If floor is made of timber, thicker joists at a greater spacing should be used, and fire stops or barriers should be provided at suitable interval.

(*v*) The flooring materials like concrete tiles, ceramic tiles, bricks etc. are more suitable for fire resistance.

(*vi*) If cast iron, wrought iron, cork carpet, rubber tiles etc. are to be used, these should be protected by a covering of insulating materials like ceramic tiles, plaster, terracotta, bricks etc.

(*vii*) Ceiling, directly suspended from floor joists should be of fire resistant materials like asbestos cement boards, fibre boards, metal lath with plaster etc.

3. Wall Openings

(*i*) From the point of view of fire spread, openings in the walls should be a bare minimum.

(*ii*) Openings serve means of escape. Hence these should be properly protected by suitable arrangements, in case of fire.

(*iii*) Doors and windows should be made of steel. Fire-resistant doors can be obtained by fixing steel plates to both the sides of the door.

(*iv*) Wire-glass panels are preferred for windows.

(*v*) Rolling shutter doors should be used for garages, godowns, shops etc.

(*vi*) In case of timber doors, minimum thickness of door leaf should be 4 cm. and that of door frame as 8 to 10 cm.

(*vii*) All escape doors should be such as to provide free circulation to the persons in passages, lobbies, corridors, stairs etc., and should be made of fire proofing material.

4. Escape Elements

(*i*) All escape elements, such as stair cases, corridors, lobbies, entrances etc. should be constructed of fire-resistant materials.

(*ii*) These escape elements should be well separated from the rest of the building.

(*iii*) Doors to these escapes should be fire proof.

(*iv*) Staircases should be located next to the outer wall and should be accessible from any floor in the direction of flow towards the exits from the building.

(*v*) Fire proof doors to the emergency stair cases should be fixed in such a way as to make them close from inside only.

(*vi*) The lift shafts connecting various floors should be surrounded with the enclosure walls of fire-resisting materials.

(*vii*) Lift shafts should be vented from top to permit escape of smoke and hot gases.

(*viii*) An emergency ladder should be provided in the fire-resisting building. This ladder should be atleast 90 cm wide, constructed of fire-resistant materials.

(*ix*) All escape routes over roofs should be protected with railings, balustrades or parapets not less than one metre in height.

5. Strong room construction

A strong room construction is found to be useful in case of safe deposit vaults in banks. Following are the important features of construction :

(*i*) The walls, floors and ceilings of a strong room are made of atleast 30 cm thick cement concrete. If thin R.C.C. walls are used, they should be have covering of bricks or terra-cotta and then suitably plastered with fire-resistant plaster.

(*ii*) Doors and windows are well anchored to concrete walls by large number of steel hold fasts longer in length.

(*iii*) Doors and windows should be fire-proof. It is preferable to have double fire-proof door.

(*iv*) Windows and ventilators should be covered by special grills made of 20 mm steel square bars. These grills should be well fixed to concrete walls by means of long steel hold fasts.

23.10. FIRE ALARMS

Fire alarms are installed to give an alarm and to call for assistance in event of fire. The fire alarms give enough time to the occupants to reach to a safe place. Fire alarms can be either manual or automatic.

1. Manual alarms

These are of a hand-bell type or similar other sounding device, which can emit distinctive sound when struck. These are sounded by watchmen and the occupants are thereby warned to have safe exit in shortest possible time. Manually operated alarms shall be provided near all main exits and in the natural path of escape from

fire, at readily accessible points which are not likely to be obstructed.

2. Automatic alarms

These alarms start sounding automatically in the event of fire. It is used in large industrial buildings which may remain unoccupied during night. The automatic fire alarm sends alarm to the nearest control point. The system can also perform the function of sending message to the nearest fire brigade station.

23.11. FIRE EXTINGUISHING EQUIPMENTS

Each building should have suitable fire extinguishing arrangements, depending upon the importance of the building and the associated fire hazards. Following are usual equipments required for fire extinction.

1. Manual fire extinguishing equipment

These devices are useful for extinguishing fire as soon as it starts. They are not so useful when once the fire has spread. Under this category comes the portable extinguishers of carbon-dioxide type or foam generation type etc. The discharge from a portable fire extinguisher lasts only for a short duration of 20 to 120 seconds. In some cases, specially in small buildings buckets of water, sand and asbestos blanket may be kept ready at all times to extinguish fire. These buckets are installed at convenient locations for taking care of fire of minor size.

2. Fire hydrants

These fire hydrants are provided on a ring main of 150 mm dia. in the ground around the building periphery. The ring main gets water from underground tank with pressure, so that available pressure at each hydrants is of the order of about 3.5 to 4 kg/cm^2.

3. Wet riser system

The system consists of providing 100 to 150 mm dia. vertical G.I. pipes (risers) at suitable locations in the building. A fire pump is used to feed water from underground tank to these pipes, to ensure a pressure of 3 kg/cm^2 at uppermost outlet.

4. Automatic sprinkler system

This arrangement is adopted for important structures like textile mills, paper mills etc. The system consists of a net work of pipes 20 mm dia. fixed to the ceiling of the room. These pipes are spaced at 3 m centre to centre. Heat actuated *sprinkler heads* are fixed to these pipes at regular interval. The pipes get supply from a header. Each sprinkler head is provided with fusible plug. In the event of fire, the fusible plug in the sprinkler nearest to the wire melts due to rise of temperature, and water gushes out of the sprinkler head. The fire is thus brought under control in a short period.

PROBLEMS

1. (*a*) What do you understand by safety of a building ?

   (*b*) Write a note on fire hazards.

2. (*a*) What do you understand by fire load ? How do you determine it ?

   (*b*) Explain how various occupancies are graded on the basis of fire load.

3. (*a*) How do you grade structural elements of the basis of fire resistance.

   (*b*) How do you grade buildings according to fire resistance?

4. Explain fire-resisting properties of various building materials.

5. Write a detailed note on fire-safety requirements for buildings.

6. Explain how do you achieve fire-resistance construction of the following elements :

   (*a*) Walls and columns.

   (*b*) Floors and roofs.

7. Write a note on 'fire escape elements'.

24

Thermal Insulation

24.1. INTRODUCTION

When there is difference in temperature of inside of a building and outside atmosphere, heat transfer takes place from areas of higher temperature to those of lower temperature. In colder regions, when the buildings are internally heated where outside atmosphere is very cool, it is necessary to check this heat loss from the building. Similarly, in very hot regions, when the buildings are internally cooled and the outside atmosphere is unbearably warm, it is essential to check the entry of heat from outside into the building. *The term thermal insulation is used to indicate the construction or provisions by way of which transmission of heat from or in the room is retarded.* The aim of thermal insulation is to minimise the transfer of heat between outside and inside of the building.

Advantages of thermal insulation

The following advantages derived from thermal insulation :

1. *Comfort.* Thermal insulation keeps the room cool in summer and hot in winter. This results in comfortable living.

2. *Fuel saving.* Since heat transfer is minimised due to thermal insulation, less fuel is required to maintain the desired temperature in the room.

3. *Prevention of condensation.* Use of thermal insulating materials inside a room results in prevention of condensation (or moisture deposition) on interior walls and ceilings etc.

4. Use of thermal insulating materials prevents the freezing of water taps in extreme winter, and heat loss in case of hot water system.

24.2. HEAT TRANSFER : BASIC DEFINITIONS

Heat transfer can take place by the following ways : (*i*) conduction (*ii*) convection, and (*ii*) radiation.

1. Conduction. Conduction is the direct transmission of heat

through a material. The amount of heat transfer by conduction depends upon (*i*) temperature difference, (*ii*) thickness of solid medium, (*iii*) area of exposed surface, (*iv*) time for which heat flow takes place, (*v*) conductivity of the medium, and (*vi*) density of the medium.

2. Convection. Heat is transmitted by convection in fluids and gases, as a result of circulation. Air movement causes the heat insulator, it is preferable to ensure that excessive air change is avoided.

3. Radiation. Heat is transferred by radiation through space in the form of radiant energy. When the radiation strikes an object, some of the energy is absorbed and transformed into heat. One of the ways of reducing heat absorption from radiation is to introduce a suitable reflecting surface.

4. Thermal conductivity (*k*). The thermal conductivity of a material is the amount of heat that will flow through an unit area of material, of unit thickness is one hour, when the difference of two temperatures is maintained at 1°C. It is expressed as $\frac{\text{k cal cm}}{\text{m}^2\,\text{h deg C}}$

Values of *k* for various building materials and insulating materials are given in Table 24.1.

5. Thermal resistivity (*l*/*k*) This is the reciprocal of thermal conductivity and is denoted by *l*/*k*.

6. Thermal conductance (*c*). It is the thermal transmission of a single layer structure per unit area divided by temperature difference between the hot and cold faces. It is expressed by $\frac{\text{k cal}}{\text{m}^2\,\text{h . deg C}}$. The values of thermal conductance of air gaps of different thickness are given in Table 24.3.

7. Thermal resistance (*R*). It is the reciprocal of thermal conductance. For a structure having plane parallel faces, thermal resistance is equal to thickness (L) divided by thermal conductivity

$$R = \frac{L}{k}$$

It is expressed as $\frac{\text{m}^2\,\text{h deg C}}{\text{k cal}}$

The usefulness of this quantity is that when heat passes in succession through two or more components of the building unit, the resistance may be added together to get the total resistance of the structure.

8. Surface coefficient (*f*). It is the thermal transmission by convection, conduction or radiation from unit area of the surface, for unit temperature difference between the surface and the surrounding medium. It is expressed as $\frac{\text{k cal}}{\text{m}^2\,\text{h deg C}}$

9. Surface resistance (1/*f*). It is the reciprocal of surface coefficient, and is expressed as $\frac{\text{m}^2\,\text{h deg C}}{\text{kcal}}$

10. Total thermal resistance (R_T). The total thermal resistance is the sum of the surface resistances and the thermal resistance of the building unit itself. Thus,

$$R_T = \left(\frac{1}{f_0} + \frac{1}{f_i}\right) + R_1 + R_2 + R_3 + \ldots\ldots\ldots$$

where f_o = Outside surface conductance

f_i = inside surface conductance

$\frac{1}{f_o}$ for walls and roofs may be taken as 0.0515. Values of $\frac{1}{f_i}$ for walls may be taken as 0.125 and that for roof as 0.171.

$R_1, R_2, R_3, \ldots$ = Thermal resistance of different materials.

The total thermal resistance is expressed as $\frac{\text{m}^2\,\text{h deg C}}{\text{k cal}}$

11. Thermal transmittance (U) Overall thermal transmittance is the thermal transmission through unit area of the given building divided by the temperature difference between the air or other fluid on either side of the building unit in 'steady state' conditions. It is reciprocal of total thermal resistance, and is expressed as $\frac{\text{k cal}}{\text{m}^2\,\text{h deg C}}$. 'Thermal transmittance' differs from 'thermal conductance' in so far as temperatures are measured on the two surfaces of material or structure in the latter case and in the surrounding air or other fluid in the former. The conductance is a characteristic of the structure whereas the transmittance depends on conductance and surface coefficients of the structure under the conditions of use. The recommended values of thermal transmittance are given in Table 24.4. The value of thermal transmittance of a structure serves as a guide for thermal insulation and the value of thermal transmittance can be brought down to the required level by adding thermal insulating material in the structure.

12. Thermal damping (D). It is expressed by the equation

$$D = \frac{T_o - T_i}{T_o} \times 100$$

where T_o = Outside temperature range

T_i = Inside temperature range.

Thermal damping or decreased temperature variation is a characteristic dependent on the thermal resistance of the materials used in the structure.

13. Thermal time constant (T). It is the ratio of heat stored to thermal transmittance of the structure

$$T = \frac{Q}{U}$$

where Q = Quantity of heat stored.

For *homogeneous* wall or roof, thermal time constant may be calculated from the following expression

$$T = \frac{Q}{U} = \left(\frac{1}{f_o} + \frac{1}{2k}\right) L \rho c$$

where f_o = Surface coefficient of outside surface
k = Thermal conductivity of the material
L = Thickness of the component
ρ = Density of the material
c = Specific heat of material.

For composite wall or roof, T may be obtained from the following expression

$$T = \sum \frac{Q}{U} = \left(\frac{1}{f_o} + \frac{L_1}{2k_1}\right)(L_1 \rho_1 c_1) + \left(\frac{1}{f_o} + \frac{L_1}{k_1} + \frac{L_2}{2k_2}\right)(L_2 \rho_2 c_2) + \left(\frac{1}{f_o} + \frac{L_1}{k_1} + \frac{L_2}{2k_2} + \frac{L_2}{2k_3}\right)(L_3 \rho_3 c_3) \quad \text{....(24.1)}$$

Typical values

Typical building constructions and the values of thermal damping (D), weight per unit area of surface for full thickness (W) thermal time constant (T) and thermal transmittance are given in Fig. 24.1 (walls) and Fig. 24.2 (roofs).

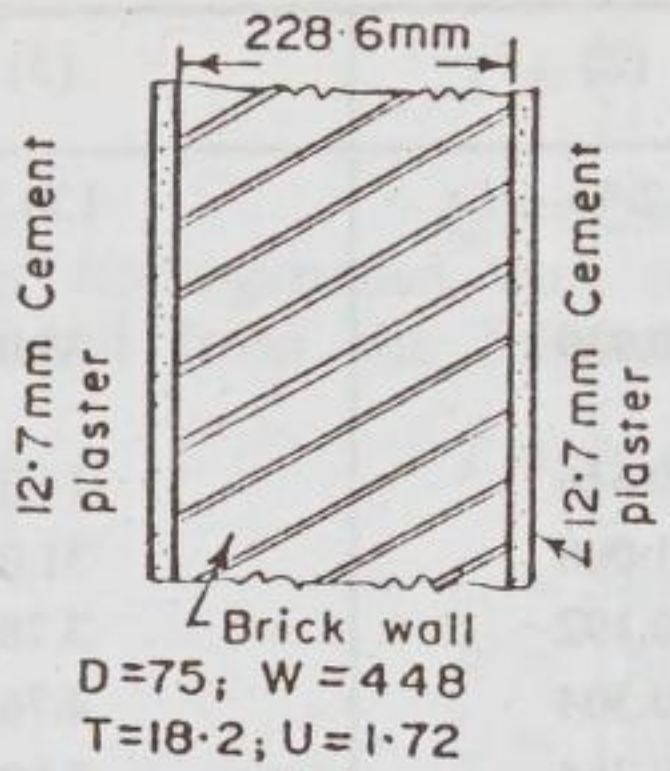
TABLE 24.1. VALUES OF THERMAL CONDUCTIVITY (k) FOR DIFFERENT BUILDING MATERIALS AND INSULATING MATERIALS

Material (1)	*Density* (2)	*Thermal conductivity (k)* $\frac{kcal\ cm}{m^2\ h\ deg\ C}$ (3)
(*a*) *Building materials*		
1. Cement mortar (1 : 3)	1.648	81.8
2. Brick work common	(1.92)	69.7
3. Asbestos cement sheeting	1.52	24.8
4. Timber (various)	0.48 to 0.72	12.4
5. Dense concrete (1: 2 : 4)	2.288	136.4
6. Cinder concrete (1 : 4)	1.406	59.5
7. Glass (*i*)	2.64	65
(*ii*)	2.35	70
(*iii*)	2.24	94
8. Roofing felt	0.80	49..6
9. Asphalt	2.24	105.2
10. Slate	2.72	161.2
11. Stone (*i*) Granite	2.64	252.0
(*ii*) lime stone	2.18	131.5
(*iii*) sand stone	2.00	111.5

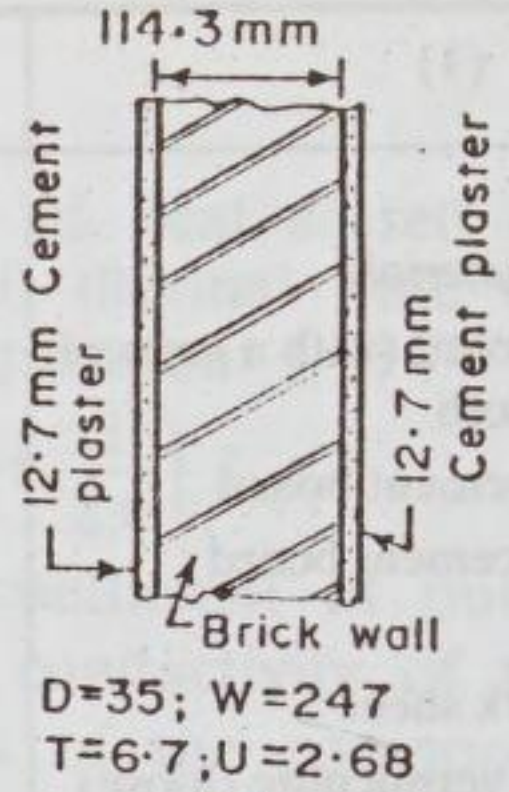
(1)	(2)	(3)
12. Terrazzo	2.43	136.3
(*b*) *Insulating materials*		
1. Gypsum board (with a layer of hessian cloth	0.939	35.0
2. Asbestos cement board	0.616	14.3
3. Asbestos cement board	1.008	31.0
4. Cork slab	0.192	3.78
5. Gasket cork sheet	0.304	4.76
6. Exfoliated vermiculite (loose)	0.264	5.99
7. Mineral wool blanket	0.192	3.35
8. Glass wool	0.189	3.47
9. Soft board (wood fibre board)	0.249	4.09
10. Wall board (wood fibre board)	0.262	4.65
11. Insulating board (laminated bitumen bounded wood fibre board)	0.342	4.77
12. Chip board	0.432	5.89
13. Chip board (perforated)	0.352	5.83
14. Foam plastic	0.042	2.73
15. Foam glass	0.160	4.79
16. Foam concrete	0.224	4.44
17. Foam concrete	0.704	12.83
18. Saw dust	0.188	4.40

TABLE 24.2. VALUES OF SURFACE CONDUCTANCES FOR VARIOUS WIND VELOCITIES

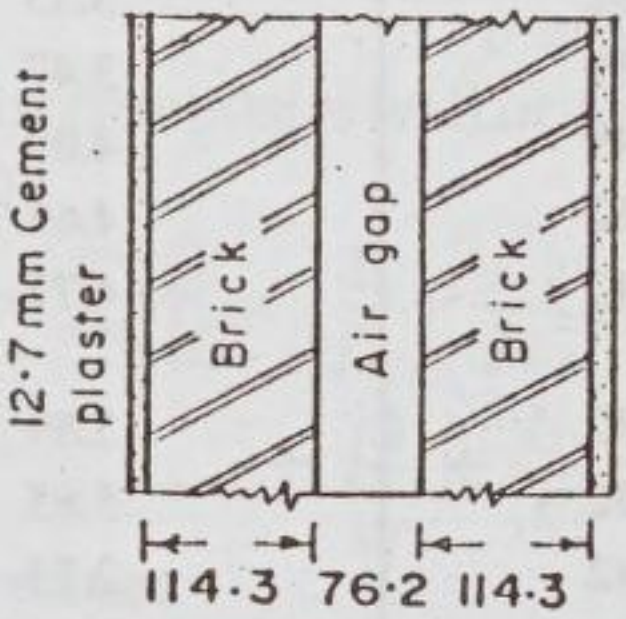
S. No. (1)	*Wind velocity* (2)	*Position of surface* (3)	*Direction of heat flow* (4)	*Surface conductance (for non reflective surface) k cal/m^2 deg C* (5)
1.	Still air	(*i*) Horizontal	Up	7.96
		(*ii*) Sloping 45°	Up	7.81
		(*iii*) Vertical	Horizontal	7.13
		(*iv*) Sloping 45°	Down	6.44
		(*v*) Horizontal	Down	5.27
2.	Moving air 24 km/hour	Any Position	Any direction (for winter)	29.29
3.	Moving air 12 km/hour	Any position	Any direction (for summer)	19.53



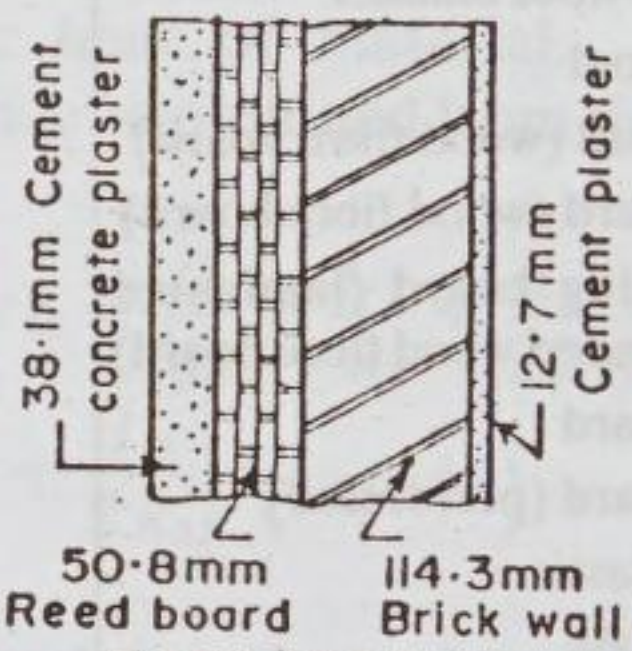
D=75; W=448
T=18·2; U=1·72
(a)



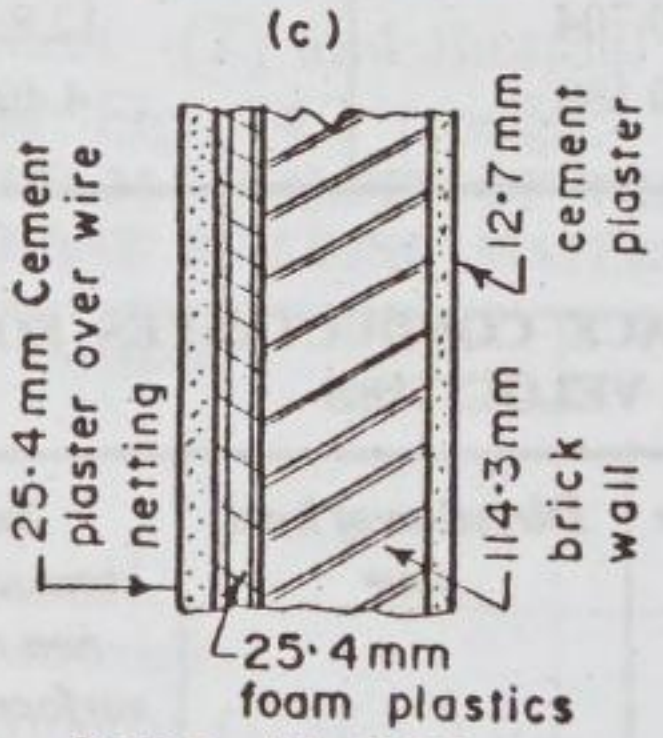
D=35; W=247
T=6·7; U=2·68
(b)



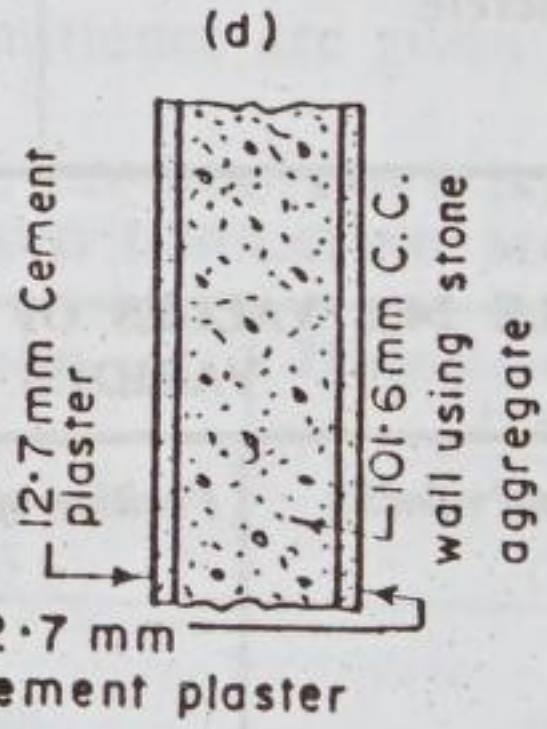
D=82; W=448
T=25·2; U=1·385
(c)



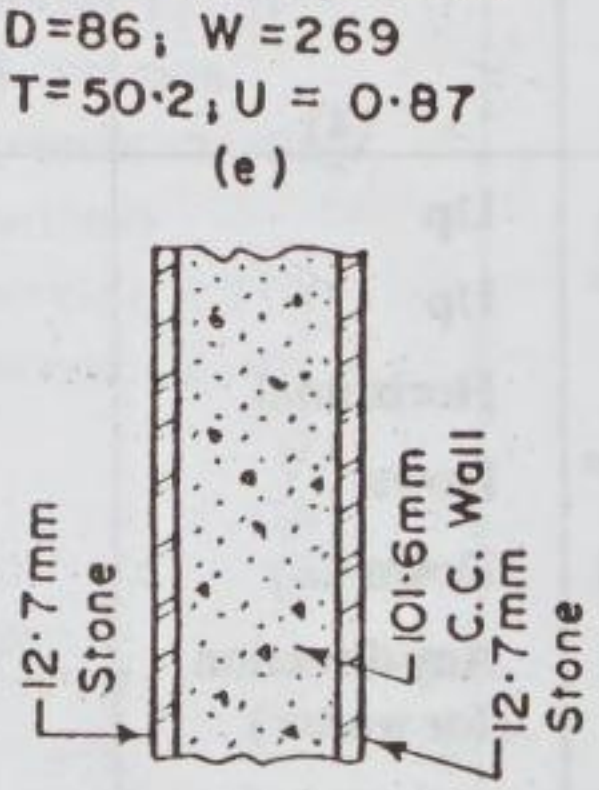
D=86; W=324
T=56·4; U=0·640
(d)



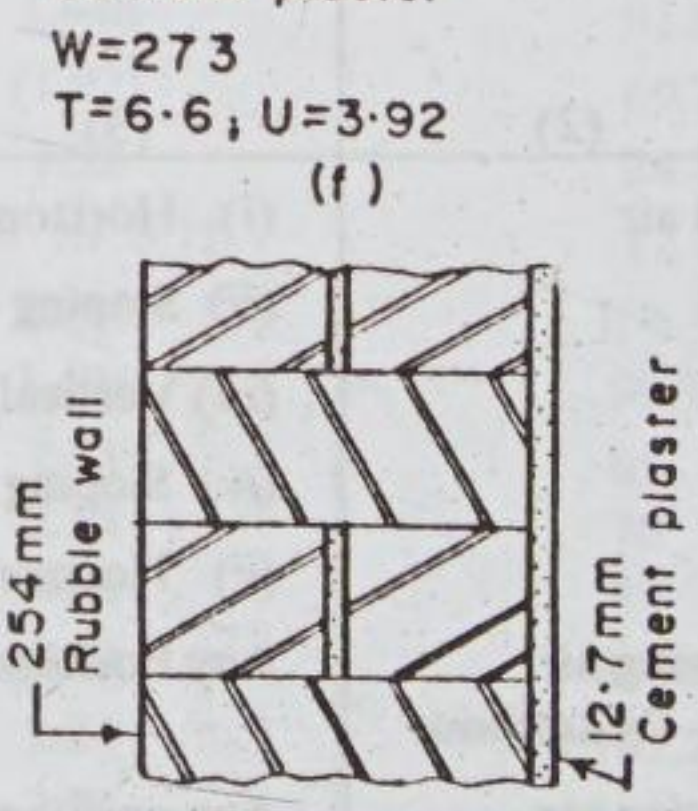
D=86; W=269
T=50·2; U = 0·87
(e)



W=273
T=6·6; U=3·92
(f)

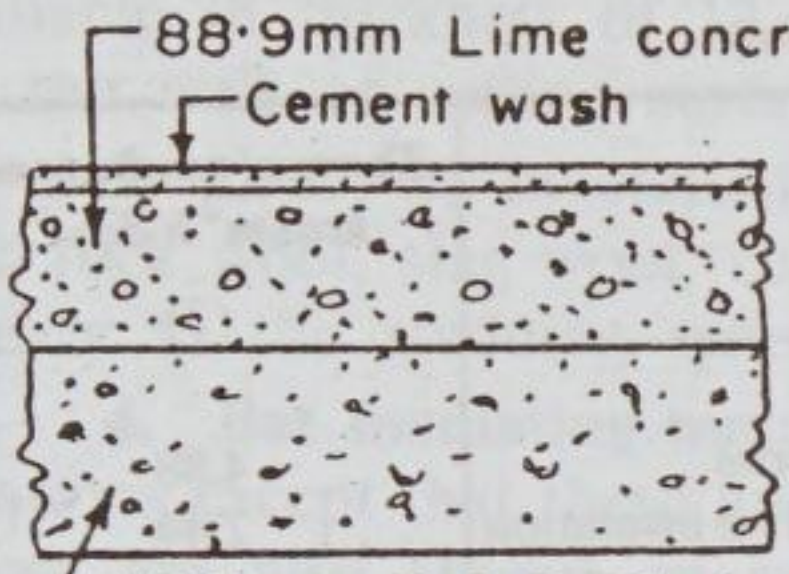


D=25; W=282
T=5·4; U=4·0
(g)

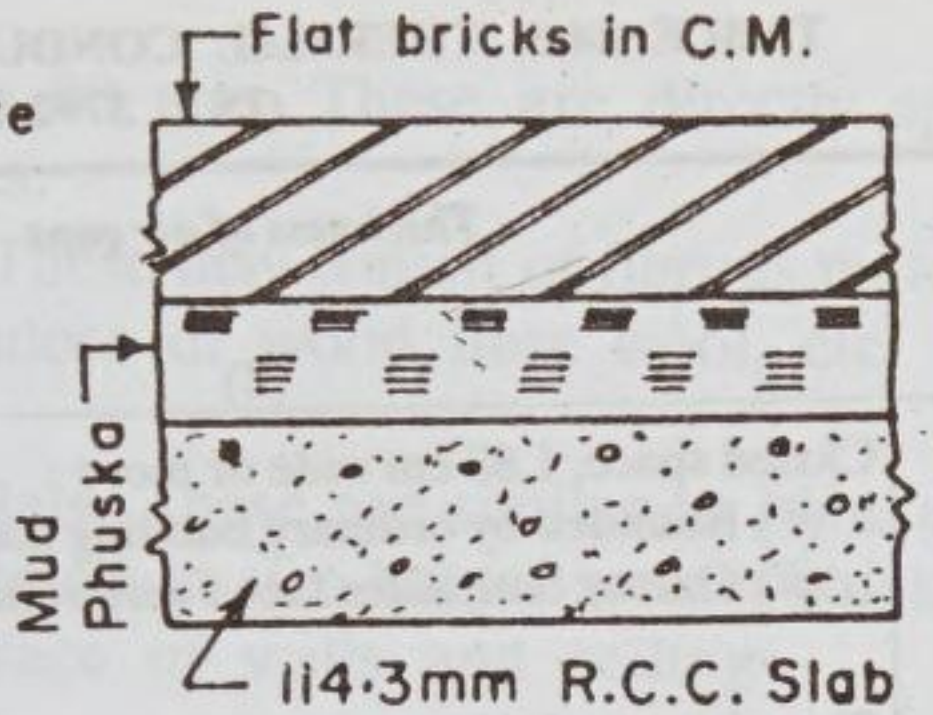


W=69
T=6·5; U=2·98
(h)

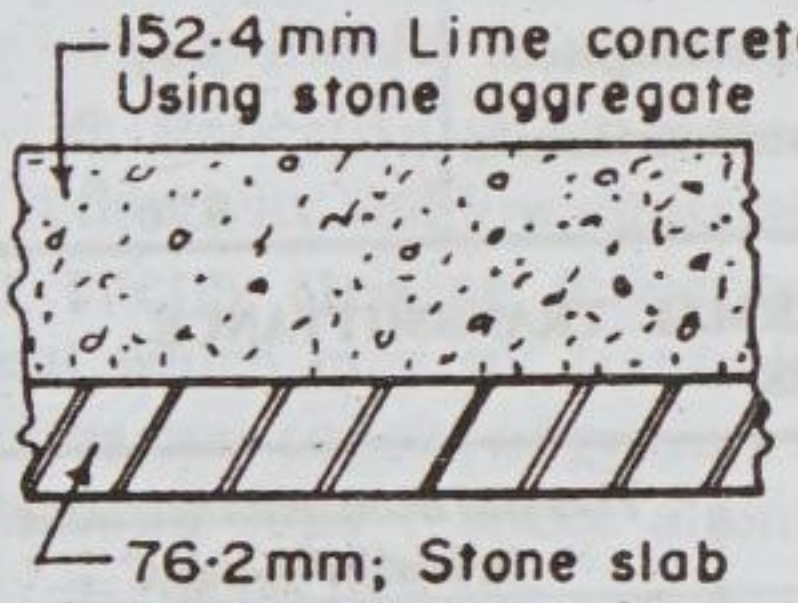
FIG. 24.1. THERMAL CONSTANTS FOR TYPICAL WALL CONSTRUCTIONS. (IS : 3792-1966)



D=63; W=394
T=14·6; U=2·54
(a)



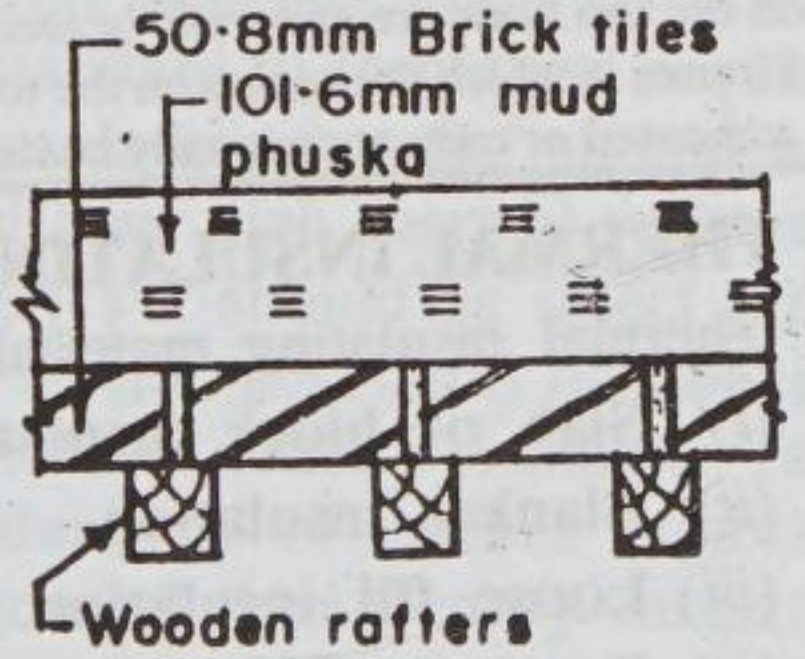
D=79; W=433
T=13·5; U=2·94
(b)



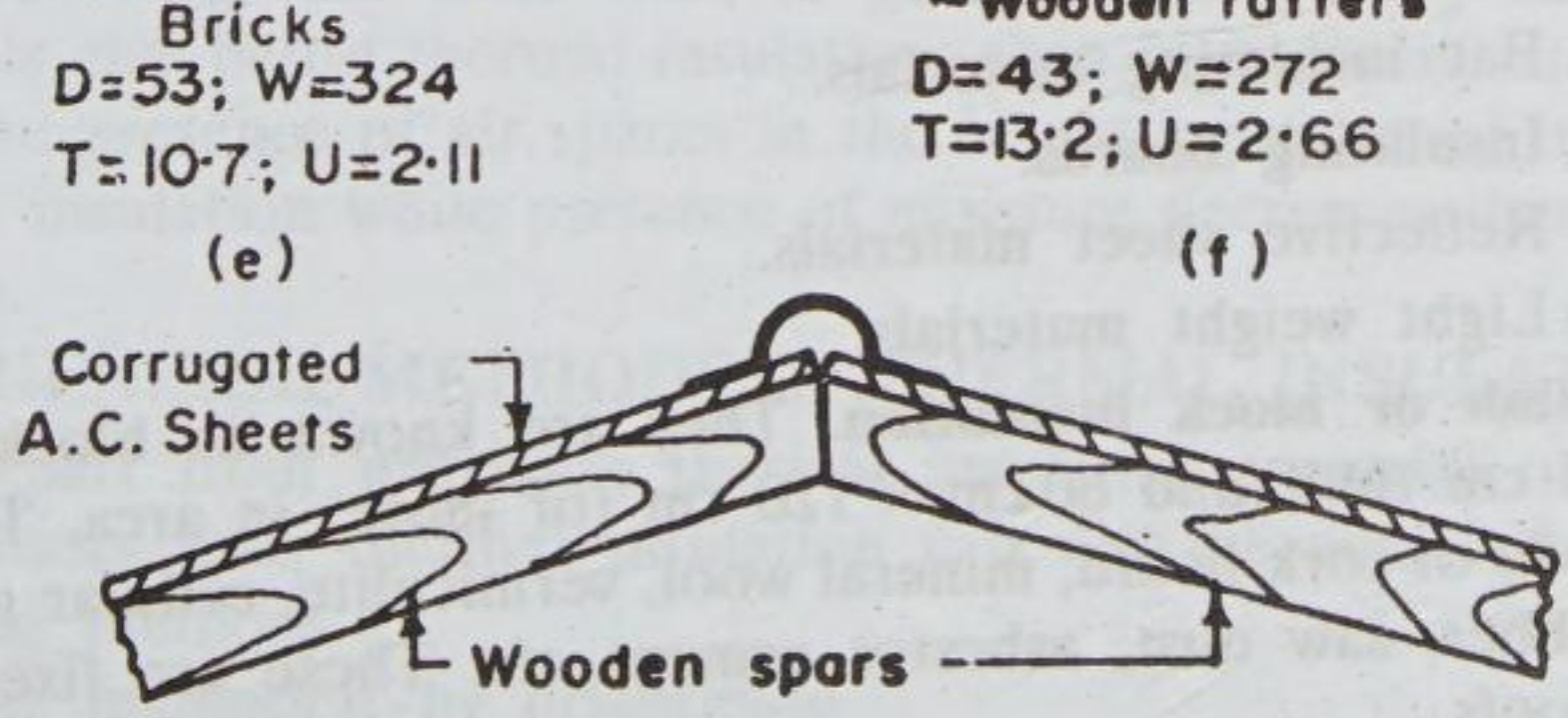
D=49; W=367
T=12·6; U=2·64
(c)

D=43; W=242
T=6·7; U=3·14
(d)

D=53; W=324
T=10·7; U=2·11
(e)



D=43; W=272
T=13·2; U=2·66
(f)



D=11; W=31; T=2·2; U=4.3
(g)

FIG. 24.2. THERMAL CONSTANTS FOR TYPICAL ROOF CONSTRUCTIONS (IS : 3792-1966)

TABLE 24.3. THERMAL CONDUCTANCE FOR AIR GAPS (IS : 3792-1966)

S.No. (1)	*Thickness of air gaps* (2)	*Thermal conductance kcal/m² h dec C* (3)
1.	Closed space, 1.88 cm wide or more :	
	(*i*) Bounded by ordinary building material	4.88
	(*ii*) One or both sides faced with reflective insulation	2.44
2.	Closed space, 0.62 cm wide :	
	(*i*) Bounded by ordinary building material	7.52
	(*ii*) One or both sides faced with reflective insulation	4.88
3.	Open space, 1.88 cm wide or more	7.52
4.	Closed space, 1.88 cm minimum, one face corrugated	5.44
5.	Closed space between plane and corrugated surfaces in contact	9.76

TABLE 24.4. RECOMMENDED THERMAL TRANSMITTANCE (U) VALUES

Surface	*Thermal transmittance value in kcal/m² h deg C*
1. External walls	1.0
2. Ground floor	1.0
3. Roof and top floor ceiling :	
(*i*) Bungalows, flats and houses in which the rooms on the top floor are generally heated	1.0
(*ii*) Houses in which the rooms on the top floor are unheated or only occasionally heated	1.5

24.3. THERMAL INSULATING MATERIALS

Thermal insulating materials may be in the following forms :

(*i*) Slab or block insulation.
(*ii*) Blanket insulation.
(*iii*) Loose fill insulation.
(*iv*) Bat insulating materials.
(*v*) Insulating boards.
(*vi*) Reflective sheet materials.
(*vii*) Light weight materials.

1. Slab or block insulation. They are known as blocks or boards, 2.5 cm thick and 60 cm × 120 cm (or more) in area. These may be made of cork board, mineral wool, vermiculite, cellular glass, cellular rubber, saw dust, asbestos cement etc. These are fixed to walls or roofs.

2. Blanket insulation. These are flexible fibrous rolls made from mineral wool, processed wood fibres, cotton, animal hair etc.,

available in thickness of 12 to 80 mm. These are directly spread on the wall or ceiling surfaces.

3. Loose fill insulation. These may consist of fibrous materials like rock wool, slag wool, cellulose or wood fibre wool, etc. filled loosely in the studding space.

4. Bat insulating materials. These are similar to blanket insulations except that these are small in size but of greater thickness. These are also spread on surface of walls and ceilings.

5. Insulating boards. These are used for interior lining of walls, and also for partition walls. Structural insulating board is manufactured by first making a pulp of wood, cane or other materials and then pressing them in form of boards by adding suitable adhesives. They are available in different sizes and thicknesses.

6. Reflective sheet materials. Reflective sheet materials have high reflectivity and low emissibility, thus offering high heat resistance. Solar energy striking reflective surfaces get reflected and amount of heat which may get transmitted is greatly reduced. Reflective insulations may consist of gypsum boards, steel sheet reflective materials, aluminium, foils ; sheet aluminium reflective materials etc.

7. Light weight aggregate. Heat resistance of concrete can be greatly increased by using light weight aggregates like blast furnace slag, burnt clay aggregates, vermiculite, etc.

Choice of insulating material

The choice of insulating material depends upon (*i*) cost of material, (*ii*) area to be covered, (*iii*) standard of insulation required, and (*iv*) coat of heating or cooling. Insulating material should have the following properties : (*a*) it should have high thermal resistance, (*b*) it should be reasonably fire proof, (*c*) it should be insect proof, (*d*) it should be durable, (*e*) it should be non-absorbant of moisture, (*f*) it should be cheaper, and (*g*) it should be readily available. Table 24.1 gives the value of thermal conductivity of various materials. It is seen from the table that, in general, low density insulating materials give better thermal insulations than high density materials. Also, the presence of air spaces in the insulating material increases thermal insulation while presence of moisture decreases thermal insulation.

24.4. GENERAL METHODS OF THERMAL INSULATION

Apart from providing thermal insulating material on walls, roofs, doors, etc., thermal insulation can also be achieved by the following methods :

1. Heat insulation by orientation

The orientation of a building with respect to the sun has a very important bearing on its thermal behaviour. For optimum orientation, there are usually conflicting requirements. Minimum trans-

fer of solar heat is desired during the day in summer, while maximum heating of rooms by solar heat is required during winter.

2. Heat insulation by shading

While shading of roof brings down the surface temperature, it is very difficult to achieve this effect in practice, especially when the altitude angle of the sun is quite high during the period of peak heat gain in afternoons, between 1100 h and 1500 h. Raising the parapet walls can help only when the altitude angle of the sun is low, but the cost may not be commensurate with the effect obtained.

3. Heat insulation by proper height of ceiling

While the surface temperature of the ceiling does not vary with its height, the intensity of long wave radiation, emitted by the ceiling decreases as it travels downwards. The effect of vertical gradient of radiation intensity is not significant beyond 1 to 1.3 m. Hence it should be adequate to provide ceiling at a height of about 1 to 1.3 m above the occupant.

24.5. THERMAL INSULATION OF ROOFS

Insulation Standards. Indian Standard, IS: 3792-1966 recommends that no roof should have an overall thermal transmittance of more than 2.00 k cal/m^2h deg C. It is also recommended that the roof should not have a thermal damping less than 75 percent (or thermal time constant less than 20 h). The relationship between *thermal damping* and *thermal time constant* is given by the limiting curves given in Fig. 24.3.

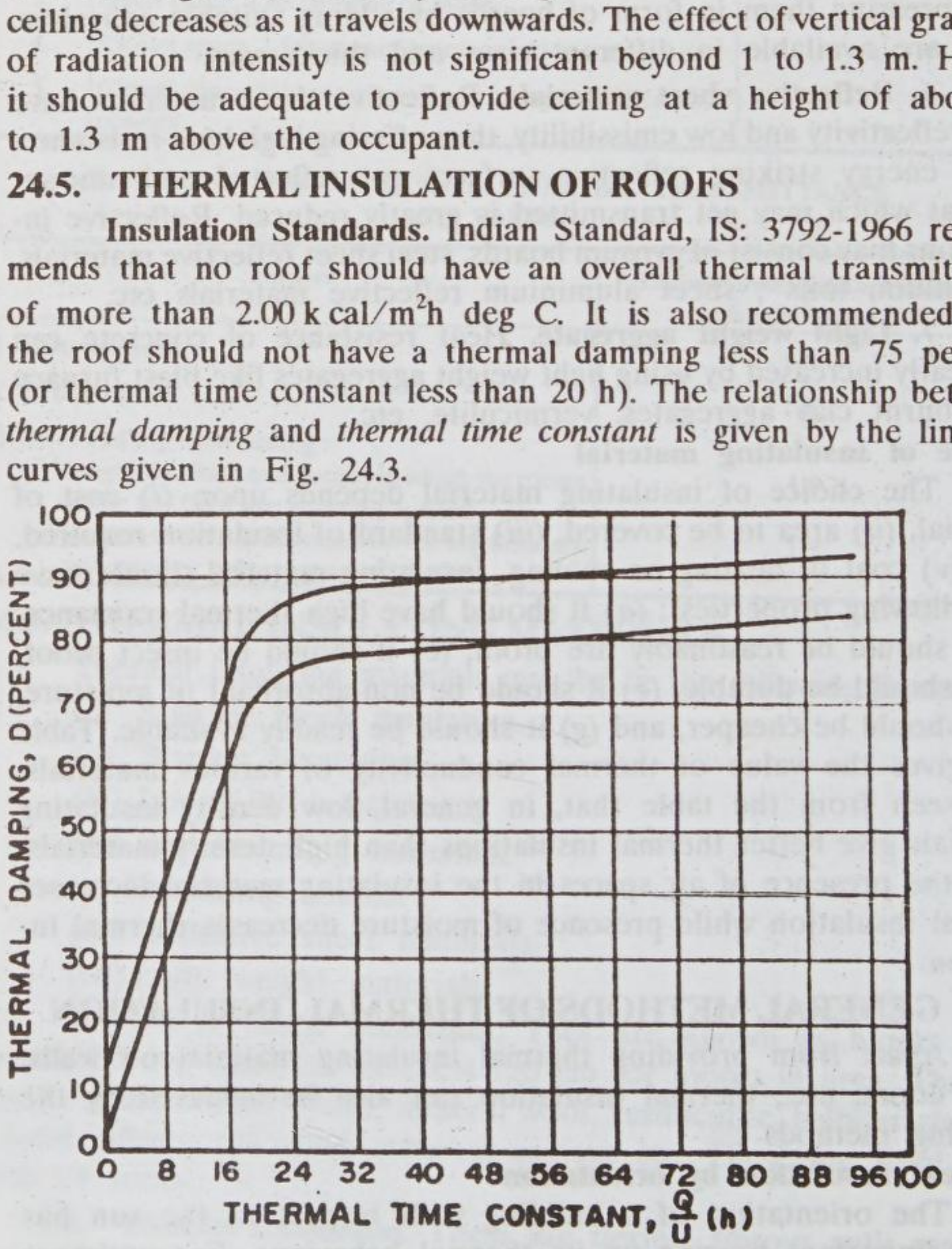


FIG. 24.3. LIMITING CURVES SHOWING RELATION BETWEEN D AND $\frac{Q}{U}$

Methods. Heat gain through roofs may be reduced by adopting the following methods :

1. *Application of heat insulating materials.* Heat insulating materials may be applied externally or internally to the roofs. In case of external application, heat insulating material may be laid over the roof but below a water proof course. In case of internal application, heat insulating material may be fixed by adhesive or otherwise on the underside of roofs from within the rooms. False ceiling of insulating material may be provided below the roof with air gaps in between, as shown in Fig. 24.4.

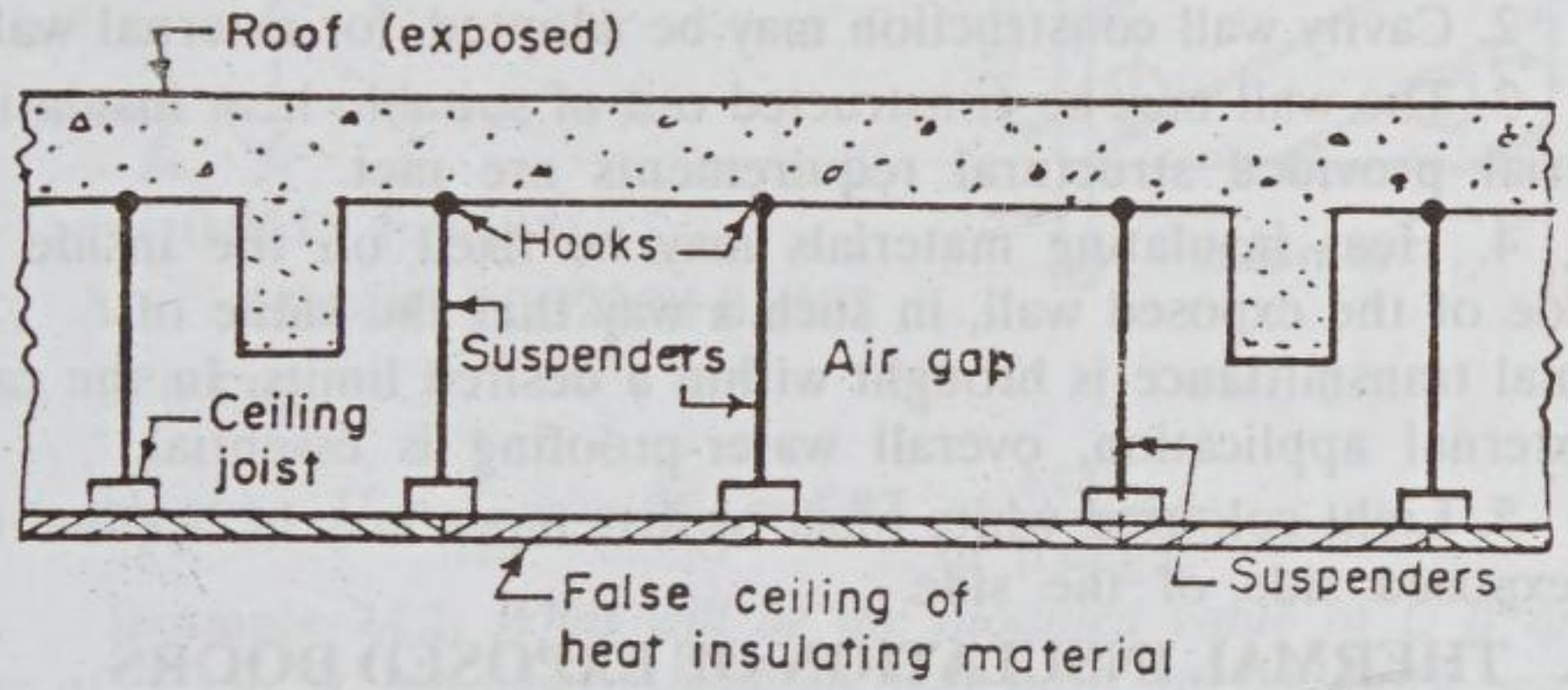


FIG. 24.4. SUSPENDED FALSE CEILING.

2. For flat roofs, external insulation may also be done by arranging asbestos cement sheets or corrugated galvanised iron sheets on bricks as shown in Fig. 24.5.

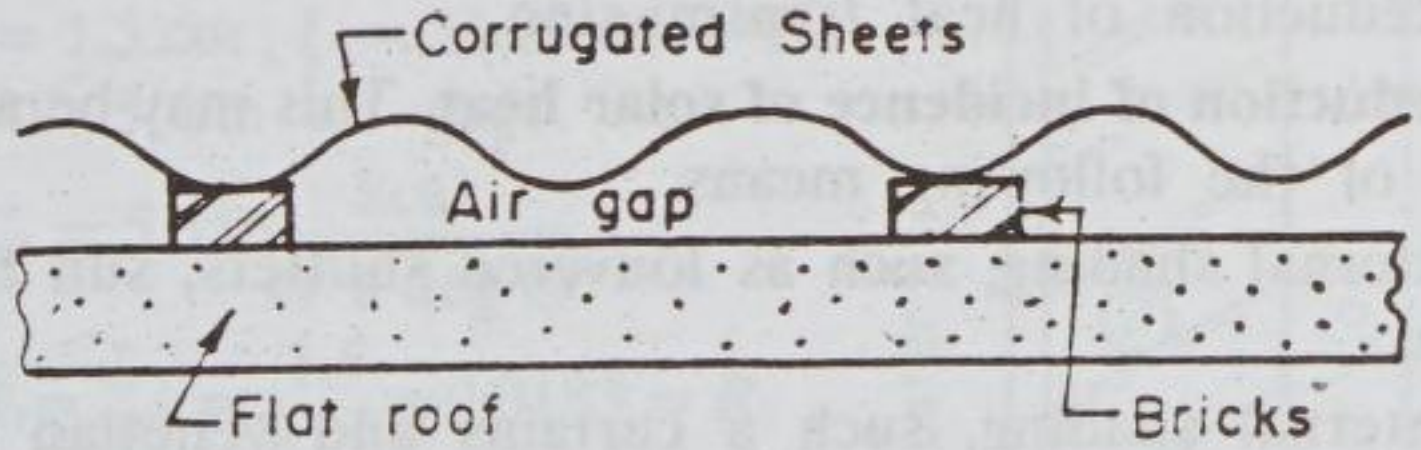


FIG. 24.5. AIR SPACE FOR FLAT ROOF.

3. Shining and reflecting materials may be fixed on the top of the roof.

4. Roofs may be flooded with water in the form of sprays or otherwise. Loss due to evaporation may be compensated by make up arrangements.

5. Roofs may be white-washed before on-set of each summer.

6. Top exposed surface of roof may be covered by 2.5 cm thick layer of coconut pitch cement concrete. Such a concrete is prepared by mixing coconut pitch with cement and water. After laying, it is covered with an impermeable layer and then allowed to dry for 20 to 30 days.

24.6. THERMAL INSULATION OF EXPOSED WALLS

Insulation Standards. IS : 3792-1966 recommends that no exposed wall should have an overall thermal transmittance of more than 2.2 kcal m^2 h deg C. It is also recommended that the wall should not have a thermal damping less than 60% (or thermal time constant less than 16 h).

Methods. Heat insulation of exposed walls may be achieved by the following ways :

1. The thickness of wall may be increased.
2. Cavity wall construction may be adopted, for external walls.
3. The wall may be constructed out of suitable heat insulating material provided structural requirements are met.
4. Heat insulating materials may be fixed on the inside or outside of the exposed wall, in such a way that the value of overall thermal transmittance is brought within a desired limits. In the case of external application, overall water-proofing is essential.
5. Light-coloured white-wash or distemper may be applied on the exposed side of the side.

24.7. THERMAL INSULATION OF EXPOSED DOORS AND WINDOWS

In dealing with heat insulation of exposed windows and doors, suitable methods should be adopted to reduce :

(*a*) Incidence of solar heat, and

(*b*) Reduction of heat transmission.

(*a*) Reduction of incidence of solar heat. This may be achieved by any one of the following means :

(*i*) External shading, such as louvered shutters, sun breakers chhajjas, and

(*ii*) Internal shading, such a curtains and venetian blinds.

(*b*) Reduction of heat transmission. Where glazed windows and doors are provided, reduction of heat transmission may be achieved by providing insulating glass or double glass with air space or by any other suitable means.

Example 24.1. *Compute the thermal transmittance (U) value for a 22.5 cm thick brick outside wall provided with 15 mm thick cement plaster on both the sides.*

Solution. (Fig. 24.6)

From Table 24.1.

$$\left.\begin{aligned} k_1 &= 81.8 \\ k_2 &= 69.7 \\ k_3 &= 81.8 \end{aligned}\right] \frac{\text{k cal cm}}{\text{m}^2\,\text{h deg C}}$$

$L_1 = 1.5$ cm ; $L_2 = 22.5$ cm ; $L_3 = 1.5$ cm

$$\therefore \qquad R_1 = \frac{L_1}{k_1} = \frac{1.5}{81.8} = 0.0183$$

$$R_2 = \frac{L_2}{k_2} = \frac{22.5}{69.7} = 0.3228$$

$$R_3 = \frac{L_3}{k_3} = \frac{1.5}{81.8} = 0.0183$$

For walls, $\frac{1}{f_i} = 0.125$ and

$$\frac{1}{f_o} = 0.0515$$

$$R_T = \frac{1}{f_o} + \frac{1}{f_i} + R_1 + R_2 + R_3$$

$$= (0.0515 + 0.125) + 0.0183 + 0.3228 + 0.0183$$

$$= 0.536$$

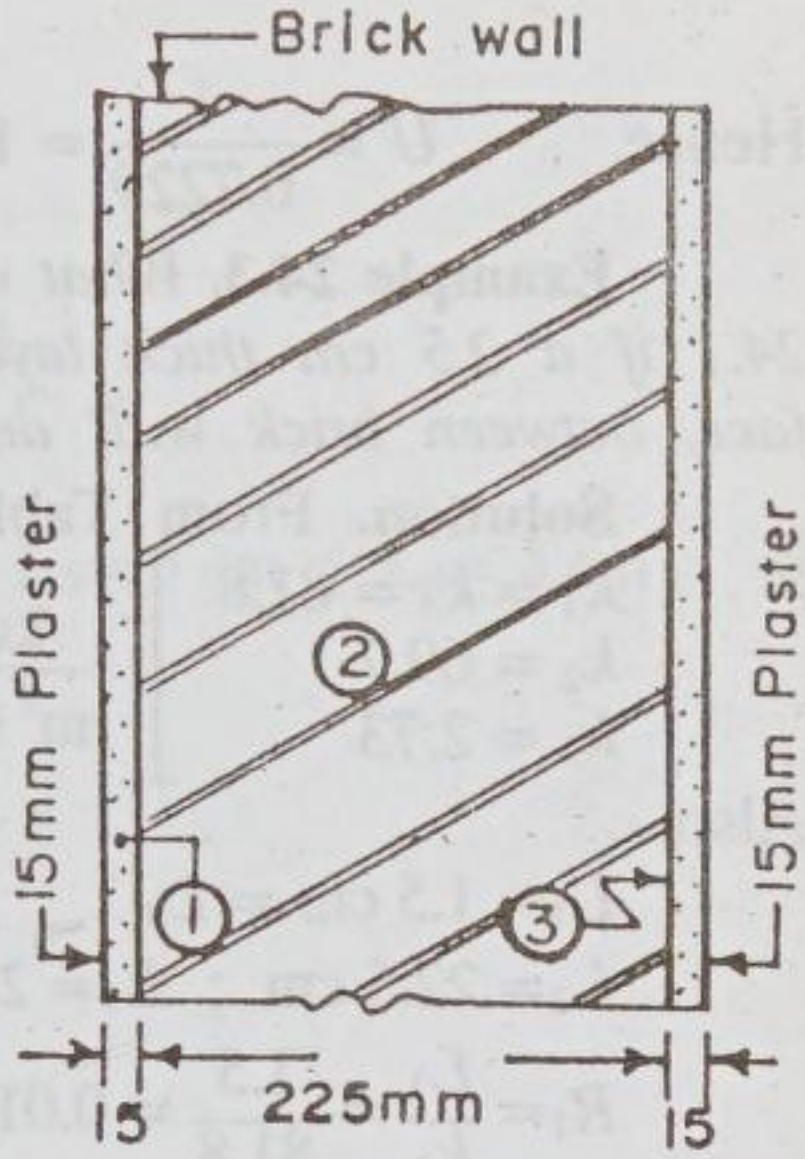


FIG. 24.6.

Hence $U = \frac{1}{R_T} = \frac{1}{0.536} \approx \mathbf{1.87} \ \frac{\text{kcal}}{\text{m}^2\,\text{h deg C}}$.

Example 24.2. *What will be the modified value of U if an air gap of 5 cm is introduced between two halfs of the brick wall of Example 24.1, as shown in Fig. 24.7.*

Solution.

$$\left.\begin{aligned} k_1 = k_5 &= 81.8 \\ k_2 = k_4 &= 69.7 \end{aligned}\right] \ \frac{\text{kcal cm}}{\text{m}^2\,\text{h deg C}}$$

$L_1 = L_5 = 1.5$ cm ; $L_2 = 11.25$ cm $= L_4$

For 5 cm air gap,

adopt $C_3 = 5.35 \ \frac{\text{kcal}}{\text{m}^2\,\text{h deg C}}$

$$R_1 = \frac{L_1}{k_1} = \frac{1.5}{81.8} = 0.0183 = R_5$$

$$R_2 = \frac{L_2}{k_2} = \frac{11.25}{69.7} = 0.1614 = R_4$$

$$R_3 = \frac{1}{C_3} = \frac{1}{5.35} = 0.187$$

Also, for wall,

$\frac{1}{f_i} = 0.125$ and $\frac{1}{f_o} = 0.0515$

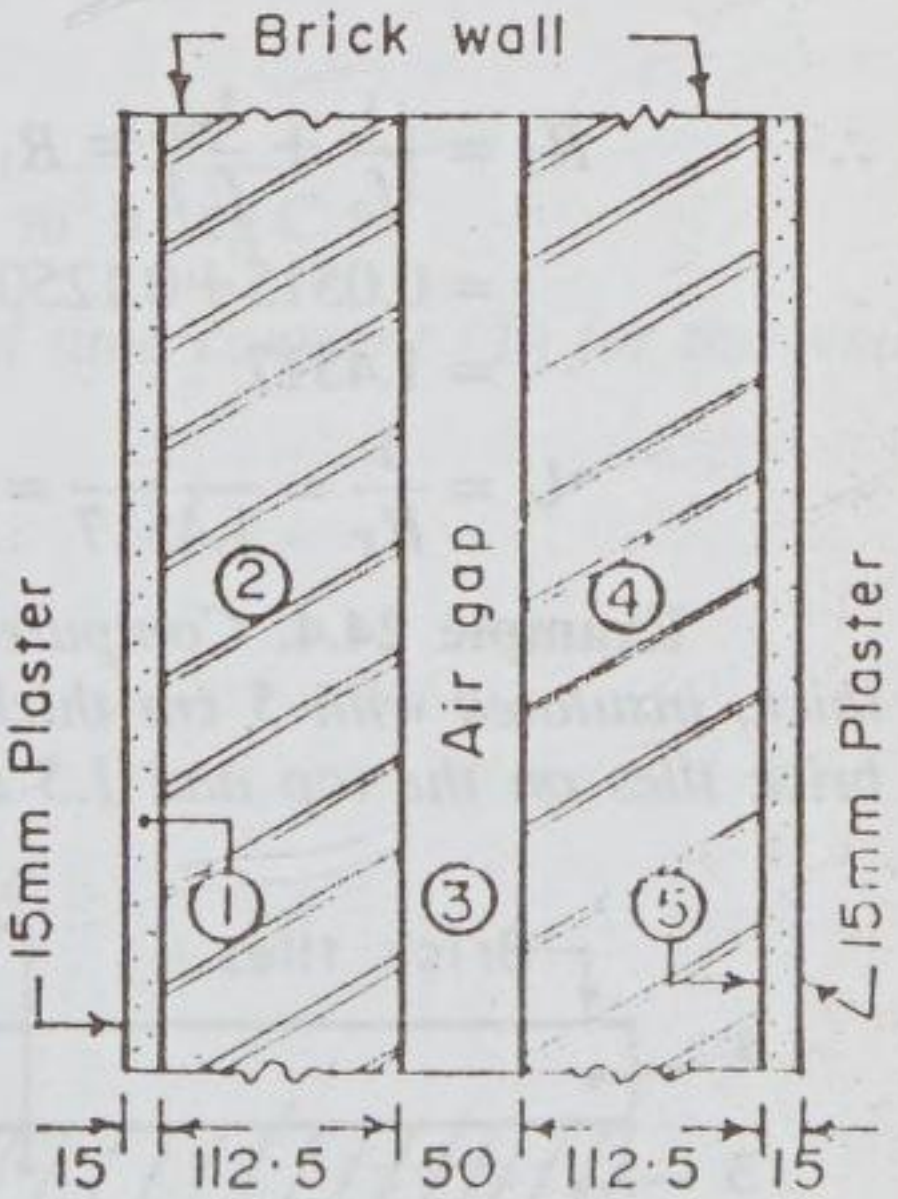


FIG. 24.7.

$$\therefore \qquad R_T = \left(\frac{1}{f_o} + \frac{1}{f_i}\right) + R_1 + R_2 + R_3 + R_4 + R_5$$

$$= (0.0515 + 0.125) + 0.0183 + 0.1614 + 0.187 + 0.1614 + 0.0183$$

$$= 0.7229$$

Hence $$U = \frac{1}{0.7229} = 1.38 \ \frac{\text{kcal}}{\text{m}^2\,\text{h deg C}}$$

Example 24.3. *What will be the U value of the wall of Example 24.1 if a 2.5 cm thick layer of foam plastic is introduced on one face, between brick wall and cement plaster ?*

Solution. From Table 24.1,

$$\left.\begin{aligned} k_1 &= k_4 = 81.8 \\ k_2 &= 69.7 \\ k_3 &= 2.73 \end{aligned}\right] \ \frac{\text{kcal cm}}{\text{m}^2\,\text{h deg C}}$$

Also

$L_1 = 1.5$ cm $= L_4$

$L_2 = 22.5$ cm ; $L_3 = 2.5$ cm

$$R_1 = \frac{L_1}{k_1} = \frac{1.5}{81.8} = 0.0183 = R_4$$

$$R_2 = \frac{L_2}{k_2} = \frac{22.5}{69.7} = 0.3228$$

$$R_3 = \frac{L_3}{k_3} = \frac{2.5}{2.73} = 0.9158$$

For wall, $\frac{1}{f_o} = 0.0515$ and $\frac{1}{f_i} = 0.125$

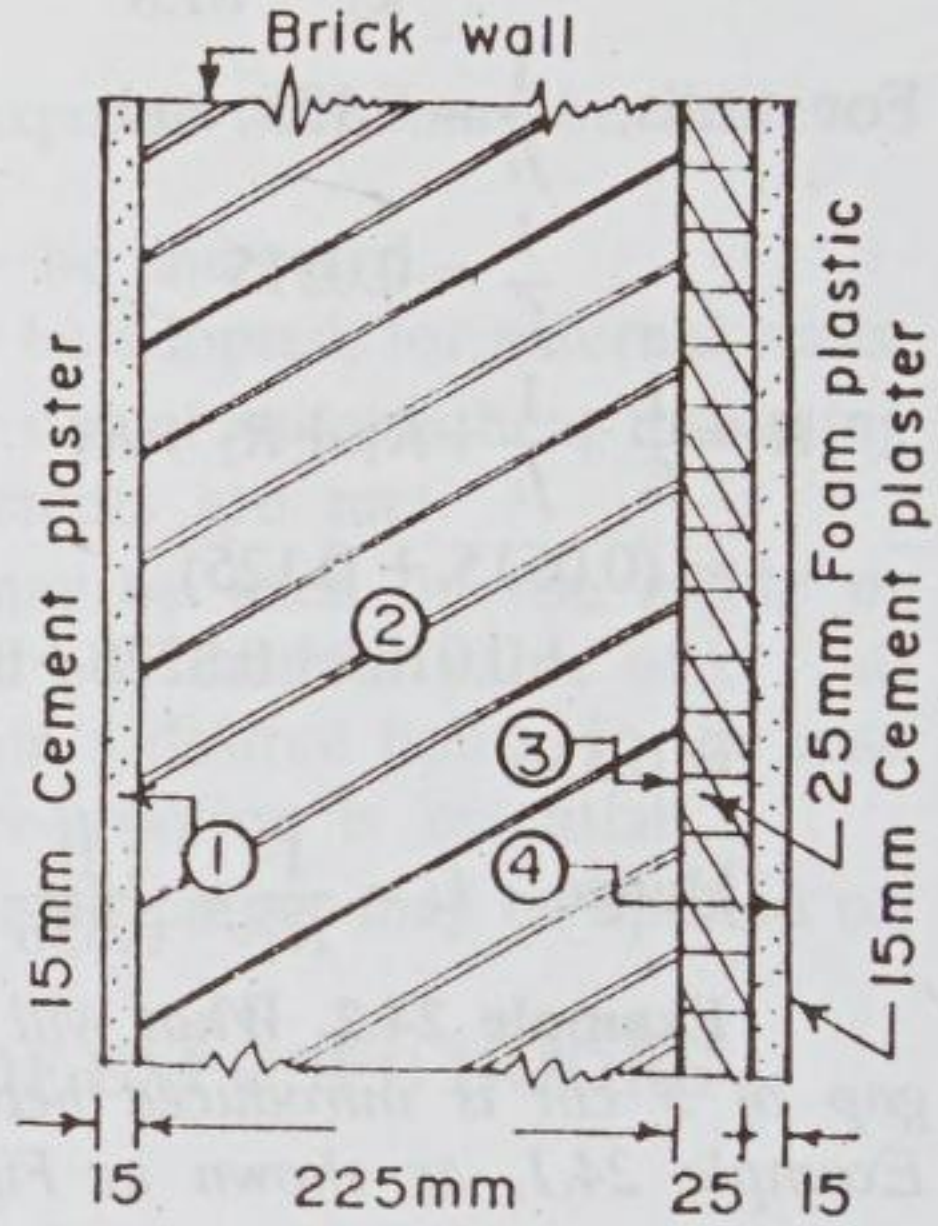


FIG. 24.8.

$$\therefore \quad R_T = \left(\frac{1}{f_o} + \frac{1}{f_i}\right) = R_1 + R_2 + R_3 + R_4$$

$$= 0.0515 + 0.1250 + 0.0183 + 0.3228 + 0.9158 + 0.0183$$

$$= 1.4517$$

$$\therefore \quad U = \frac{1}{R_T} = \frac{1}{1.4517} \approx \mathbf{0.69} \ \frac{\text{kcal}}{\text{m}^2\,\text{h deg C}}$$

Example 24.4. *Compute the U value for a R.C.C. slab, 10 cm thick, insulated with 5 cm thick foam plastic finished with 4 cm thick brick tiles on the top and 1.5 cm thick cement plaster on the bottom.*

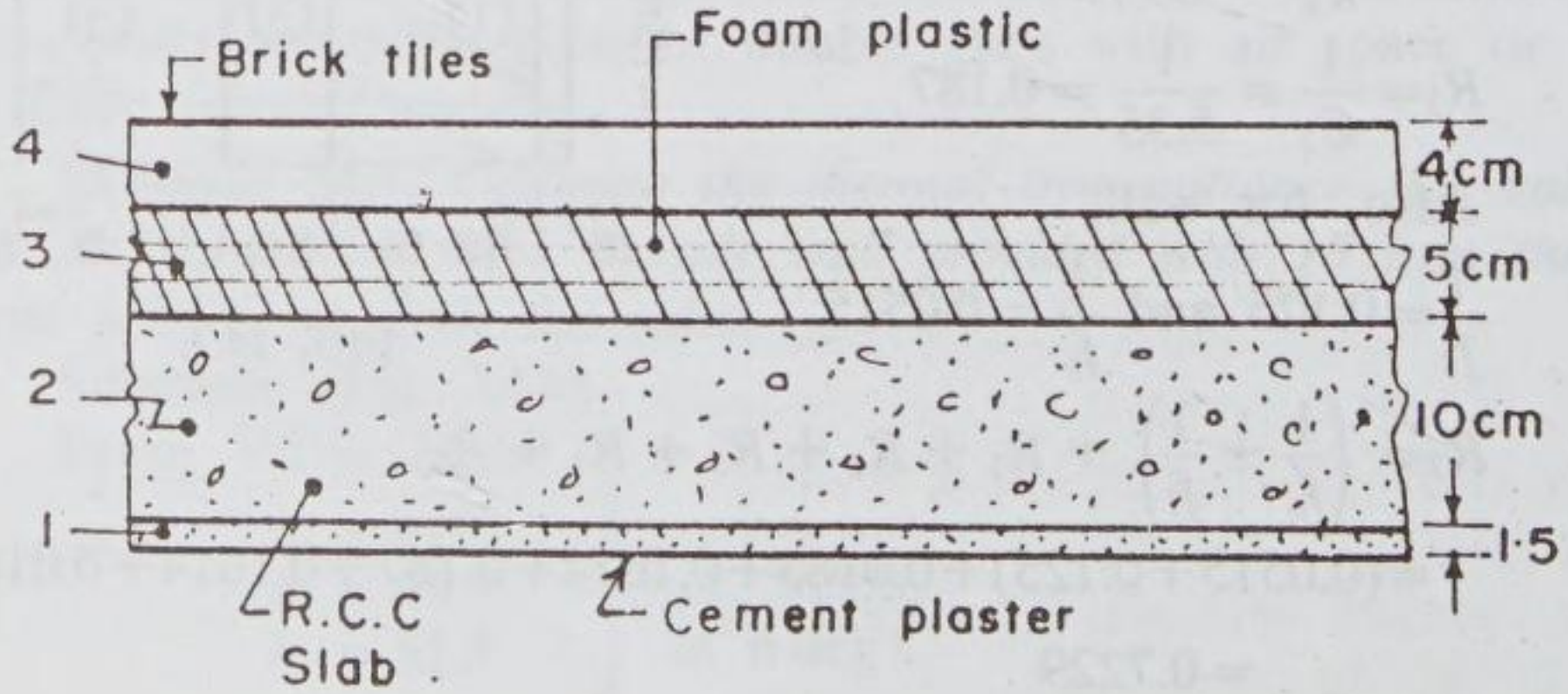


FIG. 24.9

Solution. (Fig. 24.9)
From Table 24.1, we have

$$\left.\begin{array}{l} k_1 = 81.8 \\ k_2 = 136.4 \\ k_3 = 2.73 \\ k_4 = 69.7 \end{array}\right] \quad \frac{\text{kcal cm}}{\text{m}^2\,\text{h deg C}}$$

$L_1 = 1.5$ cm ; $L_2 = 10$ cm ; $L_3 = 5$ cm ; $L_4 = 4$ cm

$$\therefore \quad R_1 = \frac{L_1}{k_1} = \frac{1.5}{81.8} = 0.0183$$

$$R_2 = \frac{L_2}{k_2} = \frac{10}{136.4} = 0.0733$$

$$R_3 = \frac{L_3}{k_3} = \frac{5}{2.73} = 1.8315$$

$$R_4 = \frac{L_4}{k_4} = \frac{4}{69.7} = 0.0574$$

For roofs, $\frac{1}{f_o} = 0.0515$ and $\frac{1}{f_i} = 0.1710$

$$R_T = \left(\frac{1}{f_o} + \frac{1}{f_i}\right) + R + R_2 + R_3 + R_4$$

$$= (0.0515 + 0.1710) + 0.0183 + 0.0733 + 1.8315 + 0.0574$$

$$= 2.2030$$

$$\therefore \quad U = \frac{1}{R_T} = \frac{1}{2.2030} = \mathbf{0.454} \quad \frac{\text{kcal}}{\text{m}^2\,\text{h deg C}}$$

Example 24.5. *Compute thermal time constant (T) for the wall of Example 24.1.*

Solution. (Fig. 24.6)

(*i*) ***For plaster,***

$L_1 = 1.5$ cm $= 0.015$ m

$k_1 = 0.818$ kcal m/m^2 h deg C

$\rho_1 = 1648$ kg/m^3

$c_1 = 0.22$ kcal/kg deg C

$$\therefore \quad L_1 \rho_1 c_1 = 0.015 \times 1648 \times 0.22 = 5.438 = L_3 \rho_3 c_3$$

$$\frac{L_1}{k_1} = \frac{0.015}{0.818} = 0.0184 = \frac{L_3}{k_3}$$

$$\frac{L_1}{2\,k_1} = 0.0092$$

(ii) For bricks

$$L_2 = 22.5 \text{ cm} = 0.225 \text{ m}$$

$$k_2 = 0.697 \text{ kcal m/m}^2 \text{ h deg C}$$

$$\rho_2 = 1920 \text{ kg/m}^3$$

$$c_2 = 0.20 \text{ kcal/kg deg C}$$

$$\therefore \quad L_2 \rho_2 c_2 = 0.225 \times 1920 \times 0.20 = 86.4$$

$$\frac{L_2}{k_2} = \frac{0.225}{0.697} = 0.3228$$

$$\frac{L_2}{2k_2} = 0.1614$$

$$T = \sum \frac{Q}{U} = \left(\frac{1}{f_o} + \frac{L_1}{2k_1}\right)\left(L_1 \rho_1 c_1\right)$$

$$+ \left(\frac{1}{f_o} + \frac{L_1}{k_1} + \frac{L_2}{2k_2}\right)\left(L_2 \rho_2 c_2\right)$$

$$+ \left(\frac{1}{f_o} + \frac{L_1}{k_1} + \frac{L_2}{k_2} + \frac{L_3}{2k_3}\right)\left(L_3 \rho_3 c_3\right)$$

$$= (0.0515 + 0.0092)\ 5.438$$

$$+ (0.0515 + 0.0184 + 0.1614)\ (86.4)$$

$$+ (0.0515 + 0.0184 + 0.3228 + 0.1614)\ (5.438)$$

$$= 0.33 + 19.98 + 3.01$$

$$= \mathbf{23.32}\ \boldsymbol{h}.$$

PROBLEMS

1. Define : Thermal conductivity, surface resistance, total thermal resistance, thermal transmittance, thermal damping and thermal time constant.

2. Explain the procedure of computing thermal transmittance and thermal time constant of a wall or roof construction of composite materials.

3. Discuss in brief various types of thermal insulating materials.

4. Explain how do you achieve thermal insulation of roofs.

5. Explain how do you achieve thermal insulation of walls.

6. Explain how do you achieve thermal insulation of exposed doors and windows.

25

Plain and Reinforced Cement Concrete

25.1. CEMENT CONCRETE

Cement concrete is a product obtained artificially by hardening of the mixture of cement, sand, gravel and water in predetermined proportions. When these ingradients are mixed, they form a plastic mass which can be poured in suitable moulds, called *forms*, and set on standing into hard solid mass. The chemical reaction of cement and water, in the mix, is relatively slow and requires time and favourable temperature for its completion. This time, known as *setting time* may be divided into three distinct phases. The first phase, designated as the time of *initial set,* requires from 30 minutes to about 60 minutes for completion. During this phase, the mixed concrete decreases its plasticity and develops pronounced resistance to flow. The second phase, known as *final set,* may vary between 5 to 6 hours after the mixing operation. During this phase, concrete appears to be relatively soft solid without surface hardness. The third phase consists of progressive hardening and increase in strength. The process is rapid in the initial stage, until about one month after mixing, at which time the concrete almost attains the major portion of its potential hardness and strength.

Depending on the quality and proportions of the ingradients used in the mix, the properties of concrete vary almost as widely as different kinds of stones. Concrete has enough strength in compression, but has little strength in tension. Due to this, concrete as such is weak in bending, shear and torsion. Hence the use of *plain concrete,* described above, is limited to applications where great compressive strength and weight are the principal requirements and where tensile stresses are either totally absent or are extremely low. However, to use cement concrete for common structures such as beams, slabs, retaining structures etc., *steel bars* may be placed at

tensile zones of the structure which may then be concreted. The steel bars, known as *steel reinforcement,* embedded in the concrete, takes the tensile stresses. The concrete so obtained is termed as *reinforced cement concrete,* commonly abbreviated as R.C.C.

25.2. CLASSIFICATION AND COMPOSITION OF CEMENT

1. Classification

Cement may be classified into five groups : (1) Portland cement (2) High Alumina Cement, (3) Super Sulphate Cement, (4) Natural Cements, and (5) Special Cements, with the following subdivisions:

1. *Portland Cements :*
   - (*a*) Ordinary Portland Cement
   - (*b*) Rapid Hardening Cement
   - (*c*) Extra Rapid Hardening Cement
   - (*d*) Low Heat Portland Cement
   - (*e*) Portland Blast Furnace Slag Cement
   - (*f*) Portland-Puzzolana Cement
   - (*g*) Sulphate Resisting Portland Cement
   - (*h*) White Portland Cement
   - (*i*) Coloured Portland Cement
2. *High Alumina Cement*
3. *Super Sulphate Cement*
4. *Natural Cements*
5. *Special Cements*
   - (*a*) Masonry Cement
   - (*b*) Trief Cement
   - (*c*) Expansive Cement
   - (*d*) Oil well Cement

2. Composition of Portland Cement

The principal raw materials used in the manufacture of cement are :

- (*a*) Argillaceous or silicates of alumina in the form of clays and shales.
- (*b*) Calcareous or calcium carbonate, in the form of lime stone, chalk and marl which is mixture of clay and calcium carbonate.

The ingradients are mixed in the proportion of about two parts of calcareous material to one part of argillaceous material and then crushed and ground in ball mills in a dry state or mixed in a wet state. The dry powder or the wet slurry is then burnt in a rotary kiln at a temperature between 1400°C and 1500°C. The clinker obtained from the kilns is first cooled and then passed on to ball mills where gypsum is added and it is ground to the requisite fineness according to the class of product.

The chief chemical constituents of Portland cement are as follows :

Lime (CaO)	60 to 67%
Silica (SiO_2)	17 to 25%
Alumina (Al_2O_3)	3 to 8%
Iron oxide (Fe_2O_3)	0.5 to 6%
Magnesia (MgO)	0.1 to 4%
Sulphur trioxide (SO_3)	1 to 3%
Soda and/or Potash ($Na_2O + K_2O$)	0.5 to 1.3 %

The above constituents forming the raw materials undergo chemical reactions during burning and fusion, and combine to form the following compounds (called Bogue compounds) in the finished product :

Compound	*Abbreviated designation*
1. Tricalcium solicate ($3CaO.SiO_2$)	C_3S
2. Dicalcium Silicate ($2CaO.SiO_2$)	C_2S
3. Tricalcium aluminate ($3CaO.Al_2O_3$)	C_3A
4. Tetracalcium alumino-ferrite ($4CaO.Al_2O_3.Fe_2O_3$)	C_4AF

The proportion of the above four compounds vary in the various Portland cements. Tricalcium silicate and dicalcium silicates contribute most to the eventual strength. Initial Portland cement is due to the tricalcium aluminate. Tricalcium silicate hydrates quickly and contributes more to the early strength. The contribution of dicalcium silicate takes place after 7 days and may continue for upto 1 year. Tricalcium aluminate hydrates quickly, generates much heat and makes only a small contribution to the strength within the first 24 hours. Tetracalcium alumino-ferrite is comparatively inactive. All the four compounds generate heat when mixed with water, the aluminate

TABLE 25.1 COMPOSITION AND COMPOUND CONTENT OF PORTLAND CEMENT (AFTER LEA)

Contents	*Normal*	*Rapid hardening*	*Low heat*
(*a*) Composition : Percent			
Lime	63.1	64.5	60
Silica	20.6	20.7	22.5
Alumina	6.3	5.2	5.2
Iron oxide	3.6	2.9	4.6
(*b*) Compounds : Percent			
C_3S	40	50	25
C_2S	30	21	45
C_3A	11	9	6
C_4AE	12	9	14

generating the maximum heat and the dicalcium silicate generating the minimum. Due to this, tricalcium aluminate is responsible for most of the undesirable properties of concrete. Cement having less C_3A will have higher ultimate strength, less generation of heat and less cracking. Table 25.1 gives the composition and percentage of the four compounds for normal, rapid hardening and low heat Portland cement.

3. Ordinary Portland Cement (IS : 269)

The properties of various types of Portland cements differ because of relative proportions of the four compounds and the fineness to which the cement clinker is ground. The *Ordinary Portland cement* or the *Setting cement* is the basic Portland cement and is manufactured in larger quantities than all the others. It is admirably suited for use in general concrete construction where there is no exposure to sulphates in the soil or in ground water.

4. Rapid hardening Portland Cement (IS : 269)

This cement is also known as *high-early strength cement.* It is similar to ordinary Portland cement except that it is ground finer, possesses more C_3S and less C_2S than the ordinary Portland cement. The magnitude of the increase in strength is gauged from the fact that the strength developed at the age of 3 days is about the same as 7 days strength of ordinary Portland cement with the same water-cement-ratio. The main advantage of a rapid hardening cement is that shuttering may be removed much earlier, thus saving considerable time and expenses. Similarly, in the concrete products industry, moulds can be released quicker. Rapid hardening cement is also used for road work where it is imperative to open the road to traffic with the minimum delay.

5. Extra Rapid Hardening Cement

Extra rapid hardening cement is obtained by inter-grinding calcium chloride with rapid Hardening Portland cement. The normal addition of $CaCl_2$ is 2% (of the commercial 70% $CaCl_2$) by weight of the rapid hardening cement. The addition of $CaCl_2$ also imparts quick setting properties. Hence this cement should be placed and fully compacted within 20 minutes of mixing.

6. Low Heat Portland Cement (IS : 269)

When concrete is poured in any structure, an increase in temperature occurs and a certain amount of heat is generated. This is due to the chemical reaction that takes place while the cement is setting and hardening. Low heat Portland cement is used in massive constructions like abutments, retaining walls, dams, etc. where the rate at which the heat can be lost at the surface is lower than at which the heat is initially generated. The heat generated in ordinary cement at the end of 3 days may be of the order of 80 calories per gram

of cement, while in low heat cement it is 50 calories per gram. It has low percentage of C_3A and relatively more C_2S and less C_3S than ordinary portland cement. This is achieved by restricting the amount of calcium and increasing the silicates present in the raw materials of manufacture. Therefore, it has low rate of gain of strength, but the ultimate strength is practically the same.

7. Portland Blast Furnace Slag Cement (IS : 455)

This cement is made by intergrinding Portland cement clinker and blast furnace slag, the proportion of the slag being not less than 25% or more than 65% by weight of cement as prescribed by IS : 455. The slag should be granulated blast furnace slag of high lime content, which is produced by rapid quenching of molten slag obtained during the manufacture of pig iron in a blast furnace. It is usual for the Portland cement clinker to be ground with a slag, a small percentage of gypsum being added to regulate the setting time. The blending of the Portland cement clinker with the slag by no means detracts from any desired property of cement. Indeed, it confers upon it some additional advantage. This is because the granulated slag itself possesses latent hydraulic properties which are tremendously activated when the slag is crystalised and integrated with Portland cement clinker. In general, blast furnace cement will be found to gain strength more slowly than the ordinary Portland cement. It has less heat of hydration than ordinary Portland cement. From the point of view of a builder and the structural engineer, Portland blast furnace slag cement may be used for all purposes for which ordinary Portland cement is used. In addition, in view of its low heat evolution, it can be used in mass concrete structures such as dams, retaining walls, foundations and bridge abutments.

8. Portland Pozzolana Cement (IS : 1489)

Portland Pozzolana cement is manufactured either by intergrinding Portland cement clinker and pozzolana or by intimately and uniformly blending Portland cement and fine pozzolana. While intergrinding presents no difficulty, *blending* tends to result in a non-uniform product and Indian Standard is specific in specifying that the latter method should be confined to factories and other such works where intimate blending can be ensured through mechanical means. As per Indian Standard, the proportion of pozzolana may vary from 10 to 25% by weight of cement. The pozzolana used in the manufacture of Portland Pozzolana cement in India is, at present, burnt clay or shale, or fly ash. Although pozzolanas have no cementing value themselves, they have the property of combining with free lime to produce a stable lime pozzolana compound which has definite cementitious properties. This cement has higher resistance to chemical agencies and to attack by sea water, because of absence

of free lime. Portland Pozzolana cement also has a lower heat of evolution. Portland Pozzolana cement is frequently stated to have lower rate of development of strength than ordinary Portland cement. However, when the puzzolana is selected with care and is calcined and ground with Portland cement clinker under controlled conditions, the compressive strengths reached by Portland pozzolana cement are comparable with those reached by ordinary Portland cement. This can be seen from the following table which compares the strength at different ages of Portland pozzolana cement and ordinary Portland cement manufactured at the cement works of the Associated Cement Companies (ACC) Ltd. India :

	Compressive Strength, N/mm²	
Age in days	*Portland Pozzolana cement*	*Ordinary cement*
3	20-22	19-23
7	26-33	27-32
28	37-48	36-52

9. Sulphate Resisting Cement

In sulphate resisting Portland cement, the quantity of tricalcium aluminate is strictly limited. They are normally ground finer than Portland cement. The action of sulphates is to form sulphoaluminates which have expansive properties and so cause disintegration of the concrete. Sulphate resisting cement should be allowed to harden in the air for as long as possible to allow a resistant skin to be formed through carbonation by the action of atmospheric carbon dioxide.

10. White and Coloured Portland Cements

The greyish colour of Portland cements is due to the presence of iron oxide. White Portland cement is manufactured is such a way that the percentage of iron oxide is limited to less 1%. To achieve this, superior raw materials, such as chalk and lime stone having low percentage of iron, and white clay(China clay) are used. Sodium aluminium fluoride(cryolite) is added to act as flux in the absence of iron oxide. Oil fuel is used in place of pulverised coal, in the kilning process in order to avoid contamination by coal ash. Coloured Portland cements are usually obtained by adding strong pigments, upto 10% to the ordinary or white cement during grinding of clinker. The essential requirements of a good pigment are that it should be permanent and should be chemically inert when mixed with cement.

11. High Alumina Cement (IS : 6452)

High alumina cement, also known as *aluminous cement foundu* is manufactured in entirely different way from that of Portland cements. The raw materials used for its manufacture are chalk and bauxite

which is a special clay of extremely high alumina content. The manufacture of this type of cement is more expensive than the Portland cements, though it has many advantages over other types of cements. High alumina cement is characterised by its dark colour, high early strength, high heat of hydration and resistance to chemical attack. It thus produces concrete of far greater strength and in considerably less time even than Rapid-Hardening Portland cement, allowing earlier removal of the form work. Its rapid hardening properties arise from the presence of calcium aluminate (chiefly monocalcium aluminate, Al_2O_3CaO) as the predominant compound in place of calcium silicates of Portland cement and after setting and hardening there is no free hydrated lime as in the case of Portland cement. However, great care should be taken in the use of high alumina cement, and it must not be mixed with any other type of cement, since the heat given off on setting is greater than with other cements.

12. Super Sulphated Cement (IS : 6909)

Super sulphated cement is made from well granulated blast furnace slag (80 to 85%), calcium sulphate (10 to 15%) and Portland cement (1 to 2%), and is ground finger than the Portland cement. One of its most important properties is its low total heat of hydration. It is, therefore, very suitable for construction of dams and mass concreting work. Concrete made from super sulphated cement may expand if cured in water, and may shrink if the concrete is cured in air. Another big advantage of super sulphated cement is its comparatively high resistance to chemical attack.

13. Natural Cements

Natural cements are those cements which are manufactured from naturally occurring *cement rocks* which have compositions similar to the artificial mix of argillaceous and calcareous materials from which Portland cement is manufactured. However, the natural cement rocks are burned at somewhat lower temperatures than those used for the production of Portland cement clinker. The properties of such cements depend upon the composition of the natural cement rock.

14. Masonry Cement

For a long time, lime gauged with sand was used for mortar for laying brick work. However, in order to increase the strength and rapidity of gaining strength, it became common to mix Portland cement wih the lime. The usual proportions of cement : lime : sand may range from 1 : 1 : 6 for heavy loads to 1 : 3 : 12 for light loads. Cement sand mortars are too harsh, while lime makes the mortar easier to work. In order to avoid the necessity for mixing cement and lime, masonry cements have recently been introduced. According to Wuerpel, most successful masonry cement are composed of Portland

cement clinker, lime stone, gypsum and air-entraining agent. These constituents are ground to an even greater fineness than that of high strength Portland cement. The plasticity and workability of masonry cements are imparted by the lime stone and air-entraining agents. The ease of working masonry cements and their water retentive properties help to increase their adhesion to bricks or other building units and this is further assisted by the fact their shrinkage is fairly low.

15. Trief Cement

Trief cement is practically the same as blast furnace cement except that the blast furnace slag is ground wet and separately from the cement. Wet grinding results in a fine product, with a specific surface of atleast 3000 cm^2/gm. Due to this, the slow rate of gain of strength normally associated with blast furnace cement is avoided and strength from early ages equal to those of ordinary Portland cement are obtained. This cement has smaller shrinkage and a smaller heat of evolution while setting than ordinary Portland cement.

16. Expansive Cement

Expansive cement expands while hardening. Ordinarily, concrete shrinks while hardening, resulting in shrinkage cracks. This can be avoided by mixing expansive cement with the normal cements in the concrete, which will neither shrink nor expand. Another useful application of expansive cement is in repair work where the opened up joints can be filled with this cement so that after expansion a tight joint is obtained. Expansive cements have been used in France for underpinning and for the repair of bomb damaged arch bridges.

17. Oil Well Cements

In the drilling of oil wells, cement is used to fill the space between the steel lining tube and the wall of the well, and to grout up porous strata and to prevent water or gas from gaining access to oil-bearing strata. The cement used may be subject to very high pressure, and the temperature may rise to 400°F. Cement used must be capable of being pumped for upto about 3 hours. It must also harden quickly after setting. These properties can be achieved by (*a*) adjusting the composition of the cement and (*b*) by adding retarders to ordinary Portland cement. In case (*a*), the proportion of Fe_2O_3 is adjusted so that it is above that required to combine with all the Al_2O_3 to form tetra calcium alumino- ferrite $4CaO.Al_2O_3, Fe_2O_3$. The proportion of tricalcium aluminate $3\ CaO.Al_2O_3$ formed is therefore very small and the setting time is accordingly increased. Setting times of upto 4 hours at a temperature of 200°F and 6 hours at a temp. of 70°F can be obtained with a Portland cement containing no tricalcium aluminate. By the use of retarders setting times of upto $6\frac{1}{2}$ hours at temperatures of upto 220°F can be obtained.

25.3. SPECIFICATIONS FOR PORTLAND CEMENT

For the quality control of Portland cement used for plain and reinforced concrete, the Indian Standard Institution has recommended the following specification and tests : (1) Chemical composition, (2) fineness (3) soundness, (4) setting time, (5) compressive strength, and (6) heat of hydration.

The specification for the above requirements, recommended by the Indian Standard are given below :

1. Chemical Requirements

When tested in accordance with the methods given in IS : 4032- 1968 (Methods of chemical analysis of hydraulic cement), ordinary cement and rapid hardening Portland cement shall comply with the following chemical requirements :

(*a*)	Ratio of percentage of lime to percentage of silica, alumina and iron oxide ; when calculated by the formula. $\frac{CaO - 0.7\,SO_3}{2.8\,SiO_2 + 1.2Al_2O_3 + 0.65Fe_2O_3}$	Not greater than 1.02 and not less than 0.66.
(*b*)	Ratio of percentage of alumina to that of iron oxide.	Not less than 0.66
(*c*)	Weight of insoluble residue	Not more than 2 percent
(*d*)	Weight of magnesia	Not more than 6 percent
(*e*)	Total sulphur content calculated as sulphuric anhydride (SO_3)	Not more than 2.75 percent
(*f*)	Total loss on ignition	Not more than 4 percent.

When tested in accordance with methods given in IS : 4032-1968, Low Heat Portland cement shall comply with the following requirements as to its chemical composition :

The percentage of lime, after deduction of that necessary to combine with the sulphuric anhydride present, shall be :

not more than $2.4(Si\,O_2)+1.2(Al_2\,O_3)+0.65(Fe_2\,O_3)$

and not less than $1.9(Si\,O_2)+1.2(Al_2\,O_3)+0.65(Fe_2\,O_3)$.

Each symbol in brackets refers to the percentage (by weight of total cement) of the oxide, excluding any contained in the insoluble residue. In all other respects, low heat Portland cement shall comply with requirements specified in (*b*), (*c*), (*d*), (*e*) and (*f*) above.

2. Fineness

When tested for fineness in terms of specific surface, Blaine's air permeability method as described in IS : 4031-1968 (methods

of chemical analysis of hydraulic cement), the cement shall comply with the following requirements :

Types of cement	*Specific surface*
Ordinary	Not less than 2250
Rapid-hardening	Not less than 3250
Low heat	Not less than 3200

3. Soundness

When tested by the 'Le Chatelier' method described in IS : 4031- 1968, unaerated ordinary rapid hardening and low heat Portland cement shall not have an expansion of more than 10 mm.

In the event of the cement failing to comply with the above requirements, a further test shall be made by the 'Le Chatelier' method, from another portion of the same sample, after aeration, by being spread out to a depth of 75 mm at a relative humidity of 50 to 80 percent for a total period of 7 days, when the expansion of each of three types of cements mentioned above shall not be more than 5 mm.

When specified by purchaser at the time of placing the order, unaerated ordinary, rapid hardening and low heat Portland cements shall not have an expansion of more than 0.3 percent when tested by the autoclave test described in IS : 4031-1968.

All cements having a magnesia content more than 3 per cent shall be tested for soundness by autoclave test and shall comply with the requirements specified in the above para.

4. Setting Time

The setting time of the cements, when tested by Vicat apparatus method shall conform to the following requirement :

	Ordinary	Rapid hardening	*Low heat*
(*a*) Initial setting time in minutes, *not less than*	30	30	60
(*b*) Final setting time in minutes, *not more than*	600	600	600

5. Compressive strength

The average compressive strength of at least three mortar cubes (area of face 50 cm^2) composed of one part of cement, three parts of standard sand (conforming to IS : 650-1966) by weight and $\frac{P}{4}+3.0$ per cent (combined weight of cement plus sand) water, and prepared, stored and tested in the manner described in IS : 4031-1968, shall be as follows :

	Ordinary N/mm^2	Rapid hardening N/mm^2	*Low heat* N/mm^2
(*a*) 24 h ± 30 min., *not less than*	—	16	—
(*b*) 72 h ± 1 h, *not less than*	16	27.5	10
(*c*) 168 ± 2 h *not less than*	22	—	16
(*d*) 672 ± 4 h. *not less than*	—	—	35

Alternatively, the cement may be accepted based on the compressive strength limits indicated in the para below :

The average compressive strength of atleast three mortar cubes(area of face 50 cm^2) composed of one part of cement, three parts of sand by weight, and P/4 + 3.5 per cent (of combined weight of cement plus sand) water, and prepared, stored and tested in the manner described in IS : 4031-1968 shall be as follows :

	Ordinary (N/mm^2)	*Rapid hardening* (N/mm^2)	*Low heat* (N/mm^2)
(*a*) 24 h ± 30 min., *not less than*	—	11.5	—
(*b*) 72 h ± 1 h, *not less than*	11.5	21	7
(*c*) 168 ± 2 h, *not less than*	17.5	—	11.5
(*d*) 672 ± 4 h, *not less than*	—	—	26.5

where *P* is the percentage of water required to produce a paste of standard consistency, to be used for the determination of the water content of mortar for the compressive strength tests and for the determination of soundness and setting time, shall be obtained by the method described in IS : 4031-1968.

6. Heat of Hydration

This requirement shall apply only to low heat cement. When tested according to the method described in IS : 4031-1968, the heat of hydration of low heat Portland cement shall be as follows:

(*a*) *7 days : not more than* 65 calories per gram, and

(*b*) *28 days : not more than* 75 calories per gram.

25.4. AGGREGATES

Aggregate is a general term applied to those inert or chemically inactive materials which, when bonded together by cement, form concrete. Most of the aggregates used are naturally occurring aggregates such as crushed rock, gravel and sand. Artificial and processed aggregates may be broken brick or crushed air-cooled blast furnace slag. Light weight aggregates, such as pumice, furnace clinker, coke, breeze, saw-dust, foamed slag, expanded clays and shales, expanded slates, expanded vermiculite etc. are also used for the production of concrete of low density.

Classification

Aggregate may be divided into two groups : (*a*) coarse aggregate,

and (*b*) fine aggregate. Aggregates less than 4.75 mm are known as *fine aggregates* while those more than 4.75 mm in size are known as coarse aggregate. For large and important works it has become usual to separate the coarse aggregate also into two or more sizes, and these fractions are kept separate until the proper quantity of each has been weighed out for a batch of concrete. *All-in aggregate,* that is to say, aggregate as it comes from the pit or river bed, is sometimes used for unimportant works.

Quality of aggregates

Natural aggregate used for concrete construction is required to comply with the norms laid down in IS : 383-1970 *'specification for coarse and fine aggregates from natural sources for concrete'*. Some of the important characteristics of aggregates are (1) strength (2) size (3) particle shape (4) surface texture (5) grading (6) impermeability (7) cleanliness (8) chemical inertness (9) physical and chemical stability at high temperatures (10) co-efficient of thermal expansion, and (11) cost.

Aggregate should be chemically inert, strong, hard, durable, of limited porosity, free from adherent coatings, clay lumps, coal, and coal residues and should contain no organic or other admixture that may cause corrosion of the reinforcement or impair the strength or durability of the concrete.

The strength of concrete depends upon the strength of aggregate. Granite aggregate provides greater strength than pumice or burnt clay aggregates. The size of coarse aggregate used depends upon the nature of work. The coarse aggregate must be small enough to enable it to be worked between and around all reinforcements and into all corners of the work. For R.C.C. work, the maximum size of aggregate is limited to 20 mm to 25 mm. A coarse aggregate may have three shapes : rounded, irregular and angular. For a concrete of given workability, rounded aggregate require least water-cement ratio while angular aggregates require highest water-cement ratio. The particle shape is thus very important, since the water-cement ratio governs greatly the strength of concrete. Similarly, a concrete made with aggregates of rough surface is stronger than with smooth one. Grading of aggregates greatly affects strength and imperviousness of concrete. If the coarse and fine aggregates are well-graded, the percentage of void is considerably reduced. The voids of the fine aggregates are then occupied by the cement paste while the voids of coarse aggregate are filled with the mortar consisting sand, cement and water. The imperviousness of aggregates is an essential requirement, especially when the concrete is used for water retaining structures. This is also essential in other R.C. works of permanence, otherwise air and moisture would penetrate with the result that outer concrete would spall out. Aggregates must be clean and free from clay, silt,

fine dust etc. so that proper mixing is possible. Dirt or other adherent coating would weaken the adhesion between the individual particles in a hardened concrete. Impurities, such as traces of sulphur or unburnt coal etc. may cause swelling due to chemical action, or may attack the reinforcement. The aggregate should have a thermal expansion similar to that of cement matrix. To summarise, the aggregate should be composed of inert mineral matter, should have high resistance to attrition, should be clean, free from any adhering coating, dense, durable and sufficiently strong to enable the full strength of the cement matrix to be developed.

Coarse aggregate

The material retained on 4.75 mm sieve is termed as coarse aggregate. Crushed stone and natural gravel are the common materials used as coarse aggregates for concrete. Natural gravels can be quarried from pits where they have been deposited by alluvial or glacial action, and are normally composed of flint, quartz, schist and igneous rocks. Coarse aggregates are obtained by crushing various types of granites (such as syenites, dolerites, diorites, quartzites etc.), schist, gneiss, crystalline hard lime stone and good quality sand stones. When very high strength concrete is required, a very fine-grained granite is perhaps the best aggregate. Coarse grained rocks make harsh concrete, and need high proportion of sand and high water/cement ratio to get reasonable degree of workability. Harder types of sand stones, having fine grained texture, are suitable as coarse aggregate, but softer varieties should be used with caution. Concrete made with sand stone aggregate gives trouble due to cracking, because of high degree of shrinkage. Similarly, hard and close-grained crystaline lime stones are very suitable for aggregate, is cheap, but should be used only in plain concrete. The bricks should be clean, hard, well-burnt and free from mortar and should not contain more than half percent of soluble sulphates. It should not be used for reinforced concrete work, since it is porous and may corrode the reinforcement. Blast furnace slag, coal ashes, coke-breeze etc. may also be used as aggregates to obtain light weight and insulating concrete of low strength.

Fine aggregate

The material smaller than 4.75 mm size is called fine aggregate. Natural sands are generally used as fine aggregate. Sand may be obtained from pits, river, lake or sea-shore. When obtained from pits, it should be washed to free it from clay and silt. Sea shore sand may contain chlorides which may cause efflorescence, and may cause corrosion of reinforcement. Hence it should be thoroughly washed before use. Similarly, if river sand contains impurities such as mud etc. it should be washed before use. Angular grained sand produces good and strong concrete, because it has good interlocking property, while round grained particles of sand do not afford such

interlocking

Grading of aggregates

Gradation of the aggregates is almost as important as its quality is. The grading of the aggregates has a marked effect on the workability, uniformity, and finishing qualities of concrete. The grading of coarse aggregate may be varied through wider limits than that of sand without appreciably affecting the workability of concrete.

Fineness Modulus

The *fineness modulus* of an aggregate is an index number which is roughly proportional to the average size of the particles in the aggregate. The coarser the aggregate, the higher the fineness modulus. The fineness modulus is obtained by adding the precentage of the weight of materials retained on the following IS sieves and dividing it by 100.

80 mm, 40 mm, 20 mm, 10 mm, 4.75 mm, 2.36 mm, 1.18 mm, 600 micron, 300 micron, and 150 micron (total 10 sieves).

Table 25.2 illustrates the method of determining fineness modulus of both coarse and fine aggregates. It has been found that certain values of fineness moduli for the fine and corase, aggregates give good workability, with a minimum quality of cement. The limits of fineness moduli are given in Table 25.3.

TABLE 25.2 DETERMINATION OF FINENESS MODULUS

IS Sieve	*Coarse aggregate (10 kg)*			*Fine aggregate (1 kg)*		
	Weight retained (kg)	*Total Wt. retained (kg)*	*% weight retained*	*Weight retained (kg)*	*Total Wt. retained (kg)*	*% weight retained*
80 mm	0.0	0.0	0.0	0.0	0.0	0.0
40 mm	0.0	0.0	0.0	0.0	0.0	0.0
20 mm	3.5	3.5	35.0	0.0	0.0	0.0
10 mm	3.0	6.5	65.0	0.0	0.0	0.0
4.75 mm	2.8	9.3	93.0	0.0	0.0	0.0
2.36 mm	0.7	10.0	100.0	0.1	0.10	10.0
1.18 mm	0.0	10.0	100.0	0.25	0.35	35.0
600 micron	0.0	10.0	100.0	0.35	0.70	70.0
300 micron	0.0	10.0	100.0	0.20	0.90	90.0
150 micron	0.0	10.0	100.0	0.10	1.00	100.0
	Sum :		693.0	Sum :		305.0
Fineness modulus	$\frac{693.0}{100} = 6.93$			$\frac{305.0}{100} = 3.05$		

TABLE 25.2. LIMITS OF FINENESS MODULI

Maximum size of aggregate		*Fineness modulus*	
		Max.	*Min..*
(*a*) Fine aggregate	—	2.0	3.5
(*b*) Coarse aggregate	(*i*) 20 mm	6.0	6.9
	(*ii*) 40 mm.	6.9	7.5
	(*iii*) 75 mm	7.5	8.0
	(*iv*) 150 mm	8.0	8.5
(*c*) Mixed aggregate	(*i*) 20 mm	4.7	5.1
	(*ii*) 25 mm	5.0	5.5
	(*iii*) 32 mm	5.2	5.7
	(*iv*) 40 mm	5.4	5.9
	(*v*) 75 mm	5.8	6.3
	(*vi*) 160 mm	6.5	7.0

25.5. WATER

Water acts as lubricant for the fine and corase aggregates and acts chemically with the cement to form the binding paste for the aggregate and reinforcement. Water is also used for curing the concrete after it has been cast into the forms.

Water used for both mixing and curing should be free from injurious amount of deleterious materials. Portable waters are generally considered satisfactory for mixing and curing of concrete. If water contains any sugar or an access of acid, alkali or salt, it should not be used. As a guide, the following concentrations represent the maximum permissible values :

(*a*) To neutralize 200 ml sample, it should not require more than 2 ml of 0.1 Normal NaOH.

(*b*) To neutralize 200 ml sample, it should not require more than 10 ml of 0.1 Normal NCl.

(*c*) Percentage of solids should not exceed the following :

	Per cent
Organic	0.02
Inorganic	0.30
Sulphates	0.35
Alkali chlorides	0.10

Carbonates and bicarbonates of sodium and potassium. Sodium carbonate may cause very rapid setting while carbonates may either accelerate or retard the setting. They may also reduce the strength of concrete, if present in large concentrations.

Chlorides and Sulphates. They are normally present in brackish water. Water is harmless if sulphates do not exceed 3000 ppm or chlorides do not exceed 10000 ppm.

Calcium Chloride. They accelerate both setting and hardening. The tolerable concentration is 2 per cent by weight of cement in

non-prestressed concrete.

Other Inorganic Salts. Salts of manganese, tin, zinc, copper and lead (nitrate) cause a marked influence on the reduction in the strength of concrete specially the last three salts are the most active. The action of lead nitrate is completely destructive. Sodium sulphide has detrimental effect and a concentration of even 100 ppm is undesirable. Salts of sodium, i.e. sodium iodate, sodium phosphate, sodium arsenate and sodium borate reduce the initial strength to a very large extent.

Turbidity. The turbidity in water, due to presence of silt, is limited to 2000 ppm.

Sea water. Sea water containing upto 3.5% salts may be used for un-reinforced concrete. However, it is undesirable to use sea water for reinforced concrete structures exposed to air, for risk of corrosion of reinforcement. Sea water should never be used for prestressed concrete.

Acidic and alkaline waters. The tolerable limits of hydrochloric, sulphuric and other common inorganic acids is 10000 ppm. Water, containing concentration of sodium hydroxide of higher than 0.5% by weight of cement may reduce the strength of concrete.

Algae. Algae in water may cause a marked reduction is strength of concrete. Algae present on the surface of aggregates also weakens the bond between them and cement paste.

25.6. MEASUREMENT OF MATERIALS

The materials used for preparation of concrete are (*i*) cement (*ii*) fine aggregate (*iii*) coarse aggregate and (*iv*) water. Their accurate measurement before mixing is very important so that the required quantities in the proportion of the concrete mix are obtained.

1. Cement. It is preferable to measure cement in terms of its weight, and not in terms of volume. The volume of cement changes with the conditions of measurement. In our country, cement is supplied in bags, each bag weighing 50 kg. Under normal conditions, the volume of cement in the bag is considered equivalent to 34.5 litres. However, if the same cement is shovelled, the bag may measure upto 42 litres. Before mixing, therefore, cement is measured in terms of weight.

2. Fine aggregates. Fine aggregate (*i.e.* sand), may be measured by weight, for accurate works and by volume for ordinary works. However, when dry sand absorbs water from atmosphere, or when water is mixed to it artificially, its volume increases. This increases in volume due to moisture in sand is known as 'bulking of sand'. Water particles lubricate the sand particles, causing surface tension, and due to this particles are pulled apart. Thus increase in volume results. This increase in volume depends on the gradation of sand,

but may be taken to be maximum at a moisture content of about 4% by weight of dry sand. Further increase in moisture results in decrease in the per cent increase of volume. The bulking increases with fineness, and may be about 25% by volume. Due to this, if sand is measured by volume, bulking should be properly accounted for.

Knowing the percentage bulking at the site, actual volume of corresponding dry sand can be estimated by subtracting from the measured volume of sand the increase in volume due to bulking. For accurate and large scale works, sand is always measured by weight and necessary allowance is made for the hygroscopic moisture in the sand.

3. Coarse aggregate. There is no problem of bulking in coarse aggregate, and hence it may be measured either by volume or by weight. However, the weight of a given volume of aggregate is influenced by the size of the measuring box. Hence for accurate and large scale works, measurement should be done by weight. The unit weight of coarse aggregate in loose and dry state is found exactly in the same manner as for fine aggregate, except that a bigger container is used. Since the size of container has effect on the determinations, Indian Standard specify the following sizes of container for carrying out the tests :

1. Maximum size of aggregate 5 mm to 40 mm : 15 litre capacity cylinder of 25 cm diameter.

2. Maximum size of aggregate over 40 mm : 40 litre capacity cylinder of 35 cm diameter.

4. Water. Water is normally measured by volume, and specified as so many litres per bag of cement. For a given quantity of water to be mixed in concrete, adjustments should be made for the amount of water present in sand and aggregate. The amount for the water present in the aggregate, due to hygroscopic action etc., should be substracted from the total required quantity of water. however, if the aggregate is dry, and found to absorb water, extra water should be *added* to account for this. The percentage absorption should be determined first.

25.7. WATER-CEMENT RATIO

Water-cement ratio is the ratio of volume of water mixed in concrete to volume of cement used. The strength and workability of concrete depend to a great extent on the amount of water used. For a given proportion of the materials, there is an amount of water which gives the greatest strength. Amount of water less than this optimum water decreases the strength and about 10 percent less may be insufficient to ensure complete setting of cement. Similarly, more than the optimum water increases the workability but decrease

the strength. An increase in 10% above the optimum may decrease the strength approximately by 15%, while an increase in 50% may decrease the strength to one-half. The use of an excessive amount of water not only produces low strength but increases shrinking, and decreases density and durability.

According to Abram's water-cement ratio law, for any given conditions of test the strength of a workable concrete mix is dependent only on the water-cement ratio. Lesser the water-cement ratio in a workable mix, greater will be its strength. From Abram's law, it follows that provided the concrete is fully compacted, the strength is not affected by aggregate shape, type or surface texture, or the aggregate grading, the workability and the richness of the mix.

According to Powers, cement does not combine chemically with more than half the quantity of water in the mix. Cement requires about 1/5 to 1/4 of its weight of water to become completely hydrated. This suggests that if water-cement ratio is less than 0.4 to 0.5, complete hydration will not be secured. Some practical values of water-cement ratio for structural reinforced concrete are about 0.45 for 1 : 1 : 2 concrete, 0.50 for $1 : 1\frac{1}{2} : 3$ concrete and 0.55 to 0.60 for 1 : 2 : 4 concrete. However, concrete vibrated by efficient mechanical vibrators require less water-cement ratio, and hence have more strength. Sometimes, plasticising agents may be mixed to increase the workability of the mix. For such concrete, therefore, water-cement ratio is reduced, resulting is an increase in the strength.

25.8. PROPERTIES AND TESTS ON CONCRETE

The important properties of concrete, which govern the design of a concrete mix are (*i*) strength (*ii*) durability, (*iii*) workability, and (*iv*) economy. The aim of proportioning a concrete mix will be to find the economic proportions of cement, coarse aggregate, fine aggregate and water so as to get a mix of a given strength, proper workability and durability.

Durability of concrete

Durability is the property of concrete by virtue of which it is capable of resisting its disintegration and decay. The concrete should be durable with proper regard to the various weathering conditions such as action of atmospheric gases, moisture changes, temperature variations. Disintegration and decay of concrete may be due to the following reasons :

(1) Use of unsound cement, which, due to some delayed chemical reactions, undergo volume changes after the concrete has hardened.

(2) Use of less durable aggregate, which may either react with cement, or may be reacted upon by atmospheric gases.

(3) Entry of harmful gases and salts through excessive pores and voids present in unsound concrete, causing its disintegration.

(4) Freezing and thawing of water sucked through the cracks or crevices, by capillary action causing its disintegration.

(5) Expansion and contraction resulting from temperature changes or alternate wetting drying.

As stated earlier, water required for chemical reaction is about 25% of the weight of cement. Hence excess water present in concrete later evaporates, leaving voids and pores. These pores or voids are later responsible for decay of concrete. Hence for durable concrete, water-cement ratio should be as small as possible to get a workable mix. A well-compacted concrete has less voids and pores and has more durability. The entrainment of air in concrete has been found to increase very considerably the resistance of concrete to freezing and thawing. The improvement in this respect is due to relief, occasioned by the minute dispersed air bubbles which act as expansion chambers, of stresses and pressures, caused by temperature and moisture changes and by the expansion of the moisture contained in concrete on freezing. Vinsol resin is sometimes mixed with concrete to have the property of entrapping innumerable minute air bubble in concrete. In order to prevent Vinsol resin reacting chemically with the cement, and to make it soluble in water, it is first neutralised by the addition of sodium hydroxide which converts it into a soap. The quantity of resin required for such purpose is extremely small—ranging from 0.005 to 0.05 of 1 percent of the weight of cement.

Workability of concrete

It is difficult to properly define and measure the 'workability' of concrete, despite its being the most important property. In its simplest form, the term 'workability' may be defined as the ease with which concrete may be mixed, handled, transported, placed in position and compacted. According to Indian Standard (IS : 1199-1959), *workability of concrete is that property of concrete which determines the amount of internal work necessary to produce full compaction.* The greatest single factor affecting the workability is the amount of water in the mix. A workable concrete does not show any bleeding or segregation. Bleeding of concrete takes place when excess of water in the mix comes up at the surface, causing small pores through the mass of concrete. Segregation is caused when coarse aggregate separate out from the finer materials, resulting in large voids, less durability and less strength.

Several tests which have been developed to measure the workability of concrete are : (1) slump test, (2) compacting factor test, (3) Vee-Bee test, and (4) Vibro-workability test.

Slump test is probably the simplest and commonly used test, though it is not the true guide to workability. In this test, concrete is compacted in a vessel of the shape of the frustum of a cone and open at both the ends. The slump test apparatus is shown in Fig. 25.1.

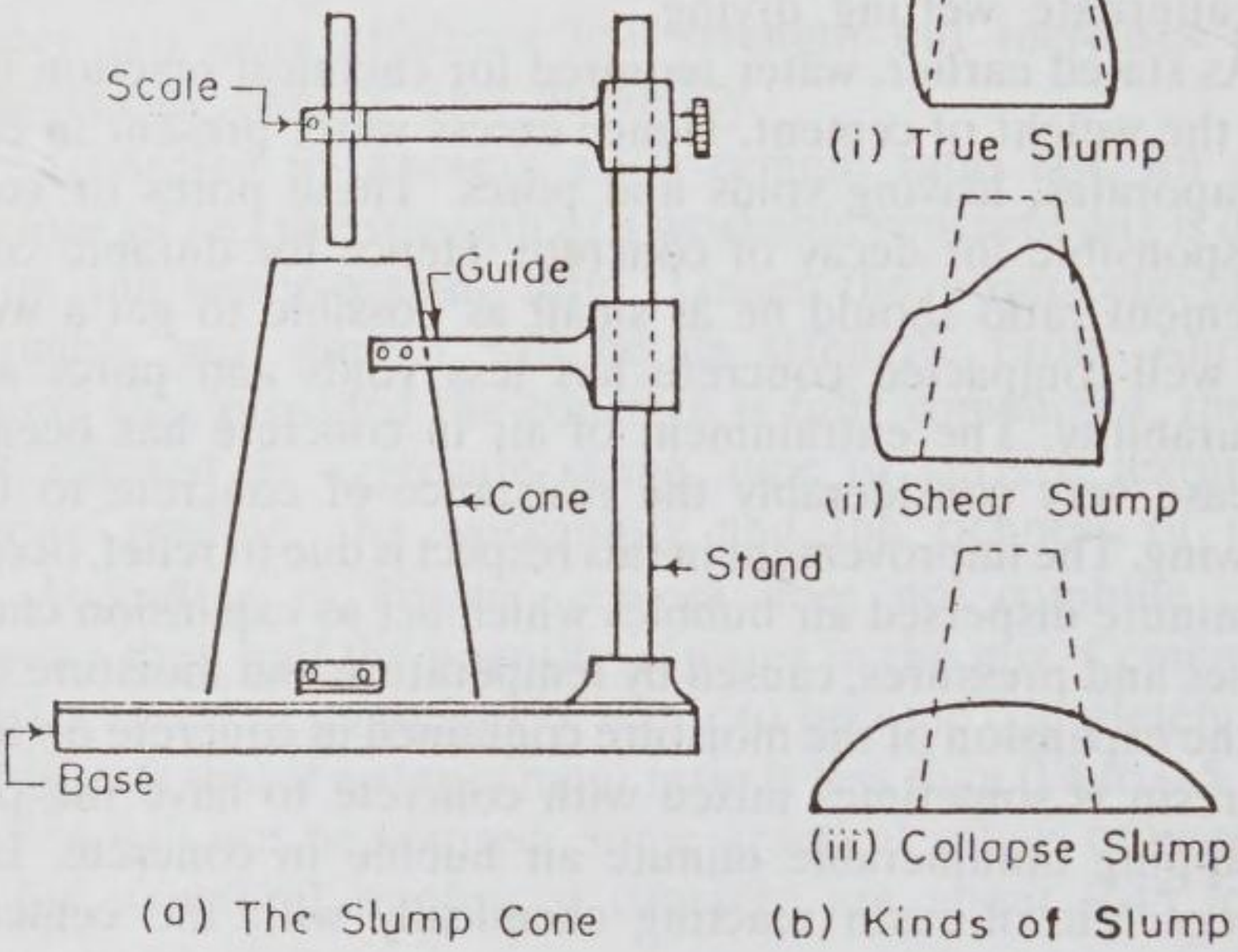


FIG. 25.1. SLUMP TEST.

To prepare the test specimen, the cleaned mould is placed on a smooth horizontal, rigid, non-absorbant surface (base). The mould is filled with freshly mixed concrete in *four* layers, each approximately one-quarter of the height of the mould. Each layer is tamped with twenty-five strokes of the rounded end of the tamping rod, the strokes being distributed in a uniform manner over the area. The strokes for the second and subsequent layers should penetrate into the underlying layer. The bottom layer should be tamped throughout its depth. After the top layer has been rodded, the concrete is struck off level with a trowel or the tamping rod. The mould is then removed from the concrete immediately by raising it slowly and carefully in the vertical direction. This allows the concrete to subside and the slump is measured immediately by determining the difference between the highest of the mould and that of the highest point of the slumped specimen. The slump measured is recorded in terms of milimeters of subsidence of the specimens during the test.

Fig. 25.1(*b*) shows three forms of slumps that may occur. The first is a *true slump*, the second is known as a *shear slump* and the third a *collapse slump* which is obtained with lean harsh or very wet mixes. Any slump specimen which collapses or shears off laterally gives incorrect results and if this occurs the test should be repeated with another sample. If in the repeat test also, the specimen should shear, the slump shall be measured and the fact that the specimen sheared, shall be recorded. Generally, if shear and collapse slumps

are obtained the concrete will be unsatisfactory for placing.

The following Table gives a rough guide of workability of concrete, in terms of slump for various works :

	Type of works	*Slump (mm)*
1	Concrete for road work	20 to 30
2	Ordinary R.C.C. work for beams and slabs ets.	50 to 100
3	Columns, retaining walls and thin vertical sections	75 to 150
4	Vibrated concrete	12 to 25
5	Mass concrete	25 to 50

The *compaction factor test* measures the workability of concrete in terms of internal energy required to compact the concrete fully. In this test, concrete is compacted in a lower cylindrical mould by making it to fall through two vertically placed hoppers. The weight of concrete in mould is determined. The theoretical weight of materials, required to fill the mould without air voids is also calculated from the knowledge of the proportions of the mix. The compacting factor is then calculated by dividing the observed weight of concrete in the mould by the theoretical weight. A concrete of low workability is represented by a compaction factor of about 0.85 of medium workability for a compaction factor of 0.92 and of good workability for a compaction factor of 0.95.

The various factors which influence the workability of concrete are (1) water in the mix (2) maximum size of particles, (3) ratio of coarse and fine aggregates, (4) particle interference. (5) particle interlocking, and (6) admixtures. Out of these, water in the mix is greatest single factor affecting the workability. Addition of water increases workability. The larger the maximum aggregate size and coarser the grading the smaller is the amount of water required for a given workability. In general, the grading requiring the least amount of water for a given workability will be that which gives the smallest surface area for a given amount of aggregate. A smooth rounded aggregate requires less water for a given workability than the irregular shaped aggregate. For a given aggregate-cement ratio, if the quantity of coarse aggregate is increased, the total surface area is reduced and hence more water would be available for lubrication, for a constant water-cement ratio, resulting in increase in workability.

25.9. METHODS OF PROPORTIONING CONCRETE MIXES

1. Arbitrary method

This method is adopted only for work of small magnitude or of moderate importance. The combined aggregate should be dense and should have least voids. For this, quantity of fine aggregates

should be sufficient to fill the voids of coarse aggregate. The ratio of coarse aggregate to fine is found to lie between $1\frac{1}{2}$ to $2\frac{1}{2}$ for a dense mix of aggregates. However, a common practice is to take the quantities of fine and corase aggregate in the proportion of 1 : 2, and hence to express the quantities of cement, sand and coarse aggregate in the proportions of 1 : n : 2n by volume. The ratios of 1 : 1 : 2 and 1 : 1.2 : 2.4 are considered suitable for very high strength concrete, the ratios $1 : 1\frac{1}{2} : 3$ and 1 : 2 : 4 are used for normal reinforced concrete work and ratios 1 : 3 : 6 and 1 : 4 : 8 are used for foundations and mass concrete work. The amount of water to be used in the above mixes is decided on the basis of workability of the mix. The workability depends upon the type of work and the method of compaction. In this method, there is no rigid control over the strength of the mix. However, because of simplicity in the design, the method is widely used for all works of small magnitude.

2. Minimum voids method

In this method, the voids of coarse aggregate and fine aggregate are determined separately. The quantity of sand used should be such that it completely fills the voids of the coarse aggregate. Similarly, the quantity of cement used should be such that it fills the voids of sand, so that a dense mix, having minimum voids is obtained. However, in actual practice, the quantity of sand used in the mix is kept 10% more than the voids in the coarse aggregate and the quantity of cement is taken 15% more than the voids in sand. To the mix of cement, sand and coarse aggregate so obtained sufficient water is added to make the mix workable. However, this method does not give satisfactory result because the presence of cement, sand and water separates the constituents of the coarse aggregate, thereby increasing its voids determined previously in absence of sand and cement. Similarly, the voids of sand are increased due to the addition of cement and water. Hence we do not always get a dense concrete. At the same time, the grading of aggregates has not been done so as to require least amount of water (and hence least w/c ratio) resulting in higher strength.

3. Maximum density method

The method of minimum voids was later improved by Fuller, to get a grading of materials to get maximum density. Based on wide scale experiments, he gave the following expression for the grading of materials :

$$P = 100 \left(\frac{d}{D}\right)^{\frac{1}{2}} \qquad ...(25.1)$$

where D = Maximum size of aggregate

P = Percentage by weight, of material finer than diameter d.

The coarse and fine aggregate should be fully graded according to the above rule. For example, let the maximum size of coarse aggregate be 20 mm and the maximum size of fine aggregate be 4 mm, the percentage of material finer than 4 mm is given by

$$P = 100\left(\frac{4}{20}\right)^{\frac{1}{2}} = 44.7\%$$

i.e. 44.7 kg of fine aggregate, including the weight of cement, are to be mixed with 55.3 kg of coarse aggregate. The quantity of various intermediate sizes should also correspond to this formula.

Let us prepare the mix having a ratio of cement to the aggregates (Fine + Coarse) as 1 : 6 by weight.

$\therefore$ Quantity of cement in 100 kg of mix $= \frac{100}{7} = 14.3$ kg

$\therefore$ Quantity of sand $= 44.7 - 14.3 = 30.4$ kg.

Hence the ratio of cement, sand and coarse aggregate by weight will be 14.3 : 30.4 : 55.3. Let us assume the unit weights of cement, fine aggregate and coarse aggregate as 1440, 1750 and 1600 kg per cubic metre respectively. Then the ratio of the three constituents, by volume will be :

$$\frac{14.3}{1440} : \frac{30.7}{1750} : \frac{55.3}{1600}$$

or $$1 : \left(\frac{30.4}{1750} \times \frac{1440}{14.3}\right) : \left(\frac{55.3}{1600} \times \frac{1440}{14.30}\right)$$

or $$1 : 1.75 : 3.48$$

or $$1 : 1\frac{3}{4} : 3\frac{1}{2} \text{ (nominally)}$$

After having decided the proportions of various materials, sufficient quantity of water is added to make the mix workable. Table 25.3 gives the grading of mixed aggregate for 40 mm and 20 mm maximum size of aggregate. The method is not so popular since grading cannot be accurately achieved in field, and there is no control over the strength.

TABLE 25.3. GRADING OF MIXED AGGREGATE

Max. Size of coarse aggregate	*Percentage passing the IS size*								
	40 mm	*20 mm*	*10 mm*	*4 mm*	*2 mm*	*1 mm*	*500 micron*	*250 micron*	*125 micron*
40 mm	100	71	50	32	22	16	11	8	6
20 mm	—	100	71	44	32	22	16	11	8

4. Fineness modulus and water-cement ratio method

(a) **Fineness modulus.** It has been observed that strength of mix is dependent wholly on the water-cement ratio while the grading of the particles is important from workability and economic point of view. The grading of particles by Fullers formula, to get maximum density, is difficult and sometimes uneconomical to achieve

in practice. Fineness modulus method essentially is a substitute for Fuller's maximum density method, aimed at standardisation of the grading of aggregates. The term *fineness modulus*, suggested by Abram, is a numerical index of fineness of both fine as well as coarse aggregates.

Certain values of fineness modulus for mixed aggregates are found to give the best result. Let p be the *desired* fineness modulus for a mix of fine and coarse aggregates. If p_1 and p_2 are the fineness moduli of fine and coarse aggregates respectively, than the proportion R of the fine aggregate to the combined aggregate, by weight is given by :

$$R = \frac{p_2 - p}{p - p_1} \times 100 \qquad ...(25.2)$$

For example, in Table 25.2 if the desired fineness modulus of the combined aggregate is 5.3, we have

$$R = \frac{6.93 - 5.3}{5.3 - 3.05} \times 100 = 72.5\%$$

(*b*) **Abram's water-cement ratio law.** Abram's water-cement ratio (w/c ratio) law states that for any given conditions of test, the strength of workable concrete mix is dependent only on the water-cement ratio. It follows from this law that provided the concrete is fully compacted, the strength is not affected by aggregate shape, type or surface texture, or the aggregate grading, the workability and the richness of mix. We know that workability of mix (defined as 'that property of the concrete which determines the amount of useful internal work necessary to produce full compaction') is dependent on the amount of water in the mix. But amount of water in the mix, corresponding to a given strength, is governed by the water-cement ratio law. Hence the only way to increase the quantity of water to increase the workability of the mix is to increase the amount of cement also. However, as the grading of aggregate does not affect the strength of the concrete directly, the object must be to choose the grading to give the best workability with the lowest water cement. The grading of the particles should, therefore, be such that in the fully compacted state the total surface area of particles of aggregates as well as voids in them are the least. This means that larger the maximum aggregate size and the coarse the grading, the smaller is the amount of water required for a given workability. However, beyond a certain limit, the further increase in the maximum size and coarseness of grading results in harsh and under-sanded mixtures causing honey combing, thus requiring more cement for smoothness. On the other hand, increase in the proportion of fine aggregate (*i.e.*, sand) gives smooth mix but requires more cement and hence

results in uneconomical mix. Between these two limits lies the optimum grading, which can be either determined by Fuller's maximum density method or the fineness modulus method.

According to Abram's law, the strength of mix increases with the decrease in the water-cement ratio. In terms of crushing strength after 7 days curing, the law can be expressed as follows :

$$p_7 = \frac{984}{7^x} \qquad ...(25.3)$$

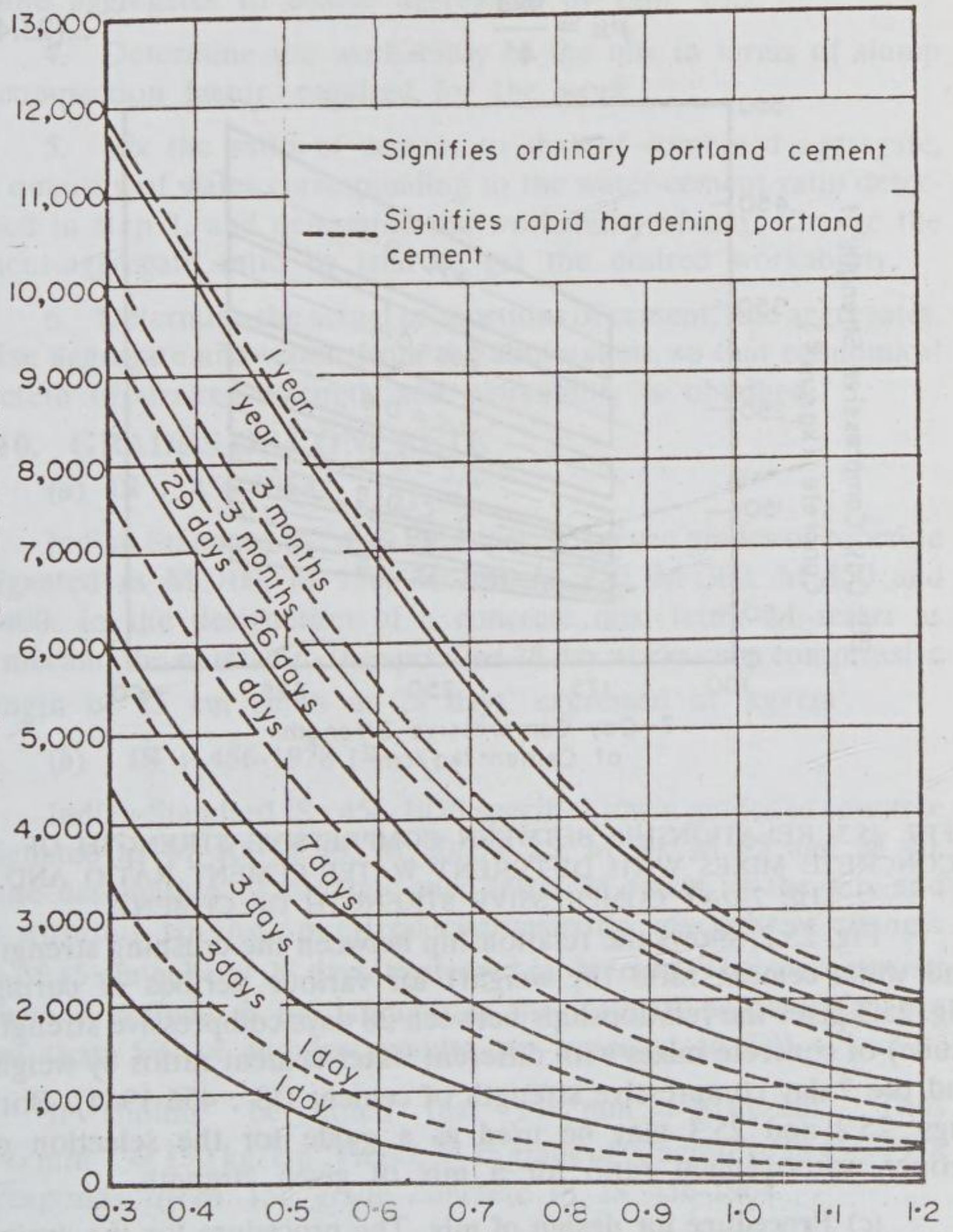


FIG. 25.2. RELATION BETWEEN CRUSHING STRENGTH AND WATER-CEMENT RATIO FOR FULLY COMPACTED CONCRETE.

where p_7= cylinder crushing strength, in kg/cm^2, after 7 days curing

x= water-cement ratio by volume.

In the above expression, the constants 984 and 7 may vary slightly with the quality of aggregates and cement, method of curing and method of testing.

Expressed in terms of strength after 28 days curing, the law can be written as

$$p_{28} = \frac{984}{4^x} \qquad ...(25.4)$$

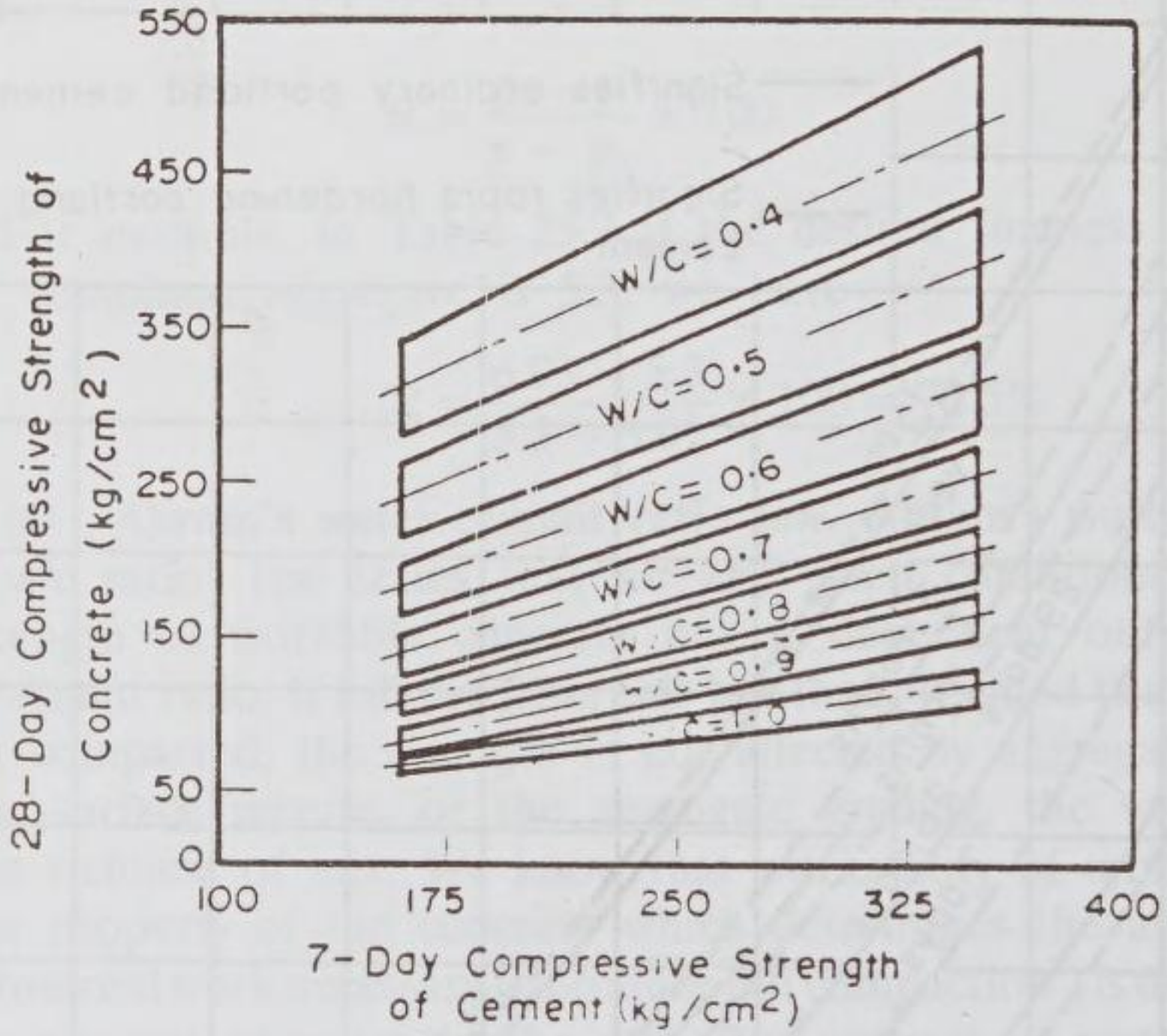


FIG. 25.3. RELATIONSHIP BETWEEN COMPRESSIVE STRENGTH OF CONCRETE MIXES WITH DIFFERENT WATER-CEMENT RATIO AND THE 7-DAY COMPRESSIVE STRENGTH OF CEMENT.

Fig. 25.2. shows the relationship between the crushing strength and water-cement ratio (by weight) for various periods of curing. Fig. 25.3 gives the relationships between 28 days compressive strength (cube) of concrete mixes with different water-cement ratios by weight and the 7-day compressive strength of cement (IS : 456-1964). Both Figs. 25.2 and 25.3 may be used as a guide for the selection of proper water-cement ratio for a mix of given strength.

(*c*) **Procedure for design of mix.** The procedure for the design of mix can be summarised as follows :

1. For the requirements of strength of the mix, choose suitable water cement ratio from Fig. 25.2 and 25.3.

2. Determine the maximum size of the aggregate available. Also determine the fineness modulus of both coarse and fine aggregates.

3. Determine the grading of aggregate by Fuller's maximum density method. If this does not correspond to the grading of available materials, try to improve the grading to make it similar to that obtained by Fuller's method. If this is difficult to achieve, design the grading by fineness mudulus method. For that, choose suitable fineness modulus of combined aggregate and determine the ratio of fine aggregates to coarse aggregates by Eqn. 25.2.

4. Determine the workability of the mix in terms of slump or compaction factor, required for the work.

5. Fix the ratio of cement to that of combined aggregate, mix quantity of water corresponding to the water-cement ratio determined in step 1, and determine the workability of mix. Change the cement-aggregate ratio by trial to get the desired workability.

6. Determine the actual proportions of cement, fine aggregates, coarse aggregate and water, from the above steps, so that economical concrete of desired strength and workability is obtained.

25.10. GRADES OF CONCRETE

(*a*) IS : 456-1964

Indian Standard IS : 456-1964 specifies seven grades of concrete designated as M 100, M 150, M 200, M 250, M 300, M 350 and M 400. In the designation of a concrete mix, letter M refers as the mix and the number to the specified 28 day works cube compressive strength of 15 cm cubes at 28 days, expressed in kg/cm^2.

(*b*) IS : 456-1978

Indian Standard IS : 456-1978 specifies seven grades of concrete designated as M 10, M 15, M 20, M 25, M 30, M 35 and M 40. In the designation of concrete mix, letter M refers as the mix and the number to the specified *characteristic compressive strength* (f_{ck}) of 15 cm cube at 28 days, expressed in N/mm^2. The *characteristic strength* is defined as the strength of the material below which not more than 5% of the test results are expected to fall.

It should be noted that $1\ N/mm^2 \approx 10\ kg/cm^2$. Thus $15\ N/mm^2 \approx 150\ kg/cm^2$. Hence M 15 grade of concrete of IS 456-1978 corresponds to M 150 grade concrete of IS 456-1964.

TABLE 25.4. GRADES OF CONCRETE

IS : 456–1964		*IS : 456–1978*	
Grade Designation	*Specified 28 days work cube compressive strength (kg/cm^2)*	*Grade Designation*	*Specified characteristic compressive strength f_{ck} at 28 days (N/mm^2)*
M 100	100	M 10	10
M 150	150	M 15	15
M 200	200	M 20	20
M 250	250	M 25	25
M 300	300	M 30	30
M 350	350	M 35	35
M 200	400	M 40	40

Note 1 : M 5 (M 50) and M 7.5(M75) grades of concrete may be used for lean concrete bases and simple foundations for masonry walls. These mixer need not be designed.

Note 2 : Grades of concrete lower than M 15 (M 150) shall not be used

PERMISSIBLE STRESSES IN CONCRETE

The permissible stresses in bending compression and direct compression, for various grades of concrete are given in Table 25.5.

TABLE 25.5. PERMISSIBLE STRESSES IN CONCRETE

Grade of concrete		*Permissible stress in compression*			
IS : 456-1964	*IS : 456–1978*	*Bending*		*Direct*	
		kg/cm^2	*N/mm^2*	*Kg/cm^2*	*N/mm^2*
M 100	M 10	30	3.0	25	2.5
M 150	M 15	50	5.0	40	4.0
M 200	M 20	70	7.0	50	5.0
M 250	M 25	85	8.5	60	6.0
M 300	M 30	100	10.0	80	8.0
M 350	M 35	115	11.0	90	9.0
M 400	M 40	130	13.0	100	10.0

Nominal mix concrete

The nominal mix concrete may be used for concrete of grades M 5 (M50), M 7.5 (M75), M 10 (M 100), M 15 (M 150) and

M 20 (M 200). The proportions of materials for nominal mix concrete shall be in accordance with Table 25.6.

TABLE 25.6.
PROPORTIONS FOR NOMINAL MIX DESIGN

Grade of concrete		*Total quantity of dry aggregate by mass per 50 kg of cement, to be taken as the sum of he individual masses of fine and coarse aggregates (max.)*	*Proportion of fine aggregate to coarse aggregate (by mass)*	*Quantity of water per 50 kg of cement (litres)*
IS : 456–1964	IS : 456–1978			
M 50	M 5	800	Generally 1 : 2 but subject to an upper limit of 1 : $1\frac{1}{2}$ and a lower limit of 1 : $2\frac{1}{2}$:	60
M 75	M 7.5	625		45
M 100	M 10	480		34
M 150	M 15	350		32
M 200	M 20	250		30

Note : The proportions of the fine to coarse aggregate should be adjusted from upper limit progressively as the aggregates become fine and the maximum size of coarse aggregates becomes larger. Graded coarse aggregates should be used.

The cement content of the mix specified in Table 25.6 for any nominal mix shall be proportionately increased if the quantity of water in a mix has to be increased to overcome the difficulties of placement and compaction, so that the water-cement ratio as specified is not exceeded.

Design mix concrete

The mix shall be designed to produce the grades of concrete having the required workability and a characteristic strength not less than appropriate values given in Tables 25.4. As long as the quality of the materials does not change, a mix design done earlier may be considered adequate for later work.

25.11. MIXING, COMPACTING AND CURING CONCRETE

1. Mixing of concrete. The operation of manufacture of concrete is called 'mixing'. The mixing can be done by two methods : (*a*) hand mixing, (*b*) mixing by concrete mixers.

(*a*) **Hand mixing.** In this method, mixing is done manually on a steel plate, 2 m × 2 m in size or on a clean hard surface. The method is resorted to when the quantity of concrete needed for the work is small. Sand and cement in appropriate proportions are mixed first in a dry state. The coarse aggregate is then added and the whole mixture is mixed thoroughly with the help of shovels.

The predetermined amount of water is then sprinkled over the mix. The mass is then turned till the colour of concrete obtained is homogeneous and workable mix is obtained.

(*b*) **Machine mixing.** Concrete should normally be mixed in a mechanical mixer. The main part of mixer is a drum in which the ingradients are mixed thoroughly by mechanically rotating the drum. The drum is made of steel plates, with a number of blades put in inclined position in the drum. As the drum rotates, the materials encounter resistance to rotation from the blades and this disturbing effect helps in a good mixing of the ingradients. The mixers are either operated electrically or else are driven by oil engines attached to them. Coarse aggregate should be fed first, then sand and lastly cement. In the revolving state, when the three get thoroughly mixed, water should be added either with the help of a can or automatically through the pipe attached to the mixer. Mixing should be contained until there is a uniform distribution of the materials and the mass is uniform in colour and consistency, but in no case shall the mixing be done for less than two minutes.

Concrete mixers may be of two types : (*i*) Batch mixers, and (*ii*) Continuous mixers. Batch type mixers are employed for work of relatively small magnitude. Batch type mixers can either be of titling drum type or closed drum type. In the tilting drum type, drum rotates about a trunnion axis and is so arranged that it is quite easy to rotate and tilt it when it is empty as well as when full. In the close drum type, the drum remains rotating in one direction and is emptied by means of the hopper which tilts to receive the discharge. Continuous mixers are used in mass concreting work where a large and continuous flow of concrete is required. In these mixers, processes of feeding, mixing and emptying go on continuously without break.

2. Transporting concrete. Concrete should be handled from the place of mixing to the place of final deposit as rapidly as practical by methods which will prevent the segregation or loss of any of the ingradients. If the segregation does occur during transport, the concrete should be remixed before being placed. During hot or cold weather, concrete should be transported in deep containers, on account of their lower ratio of surface area to mass, reduce the rate of loss of water by evaporating during hot weather and loss of heat during cold weather.

3. Placing concrete. The concrete should be placed and compacted before setting commences and should not subsequently be disturbed. Method of placing should be such as to preclude segregation. Before concrete is placed, it should be ensured that the forms are rigid, in their correct position, well cleaned and oiled. Oiling of

the forms will prevent the concrete from sticking to it, and it will then be easier to remove the forms when they are no longer required. Concrete should not be poured into the forms only at one point, but should be uniformly spread on all the sides for better compaction. When the work has to be resumed on a surface which has hardened, such surface should be roughened. It should then be swept clean, thoroughly wetted and covered with a 13 mm layer of mortar composed of cement sand in the same ratio as the cement and sand in the concrete mix. This 13 mm layer of mortar should be freshly mixed and placed immediately before the placing of the concrete. Where the concrete has not fully hardened, all lattance should be removed by scrubbing the wet surface with wire or bristle brushes, care being taken to avoid dislodgement of particles of aggregate. The surface should be thoroughly wetted and all free water removed. The surface should then be coated with neat cement grout. The first layer of concrete to be placed on this surface should not exceed 150 mm in thickness, and should be well-rammed against old work, particular attention being paid to corners and close spots.

4. Compacting concrete. The removal of entrapped air during production of concrete and the uniform, dense arrangement of the constituents of concrete are effected during the compacting of corners. The density and, consequently, the strength and durability of concrete depend upon this operation. Concrete should be thoroughly compacted during the operation of placing and thoroughly worked around the reinforcement, around embedded fixtures and into corners of the form work. Concrete is compacted by vibration, during which the vibrator communicates rapid vibrations of low amplitude to the particles, as a result of which the concrete becomes fluid, that is to say, its mobility is increased, and the particles, in movement, under the force of gravity occupy a more stable position, with which volume of concrete is least. Vibrators are of three general types (*i*) internal vibrators (*ii*) external vibrators, and (*iii*) surface vibrators. Internal or immersion vibrator consists of a vibrating element enclosed in a casing which is immersed in fresh concrete and transmit vibrations through the vibrator body. External or form vibrators are fastened to the form work by a clamping device and transmit vibrations to the concrete through the form. In precast members of concrete, the vibrating tables are very helpful. These tables vibrate the entire mass of concrete uniformly. Surface vibrators, set up on the concrete surface after placing, transmit vibrations to it through a working platform. They are generally employed in concrete road construction.

Mechanical method of compacting the concrete is used only when the mix is stiff. Over-vibration or vibration of very wet mixes is harmful and should be avoided. Alternatively, concrete may be compacted manually by rodding, temping or hammering. Rodding

is generally done to compact thin vertical members while temping is done for compacting concrete for slabs etc.

5. Curing concrete. Curing is one of the most essential operation in which concrete is kept continuously damp for some days to enable the concrete to gain more strength. Curing replenishes the loss of moisture from the concrete due to evaporation, absorption and heat of reactions. The period of curing depends upon atmospheric conditions such as temperature, humidity and wind velocity. The normal period is between 7 and 10 days. There are several methods of curing the concrete, the more common being the following : (*i*) Covering the exposed surface with a layer of sacking, canvas, hessian or similar absorbant materials, and keeping them continuously wet, (*ii*) throughly wetting the surface of concrete, and then keeping it covered with a layer of suitable water proof material, (*iii*) impounding water in earthen or sandy bunds in squares over the flooring, (*iv*) curing with the help of steam or hot water, resulting in rapid development of strength.

25.12. STEEL REINFORCEMENT

Steel reinforcement used in reinforced concrete may be of the following types :

(*a*) 1. Mild steel bars conforming to IS : 432 (Part 1) - 1966.
2. Hot rolled mild steel deformed bars conforming to IS: 1139-1966.

(*b*) 1. Medium tensile steel conforming to IS : (Part 1)-1966.
2. Hot rolled medium tensile steel deformed bars conforming to IS : 1139-1966.

(*c*) 1. Hot rolled high yield strength deformed bars (HYSD bars) conforming to IS : 1139-1966.
2. Cold-worked steel high strength deformed bars conforming to IS : 1786-1979 (grade Fe 415 and Grade Fe 500).

(*d*) 1. Hard drawn steel wire fabric conforming to IS : 1566-1967.
2. Rolled steel made from structural steel conforming to IS : 226-1975.

The permissible stresses in steel reinforcement, as per IS : 456-1978 are given in Table 25.7, in which columns 2, 3 and 4 correspond to types (*a*), (*b*) and (*c*) respectively, mentioned above. The modulus of elasticity E for these steels may be taken as 2×10^5 N/mm^2 (2×10^6 kg/cm^2).

A *twisted bar* has considerable increased yield stress, about 50% more than that of ordinary mild steel bar. Their use can permit higher working stress and hence considerable saving in quantity of steel can be achieved. Bond between concrete and steel can be improved by use of *deformed bars.* A deformed bar is a bar of steel provided with lugs, ribs or deformations on surface of the bar to minimize the slippage of the bar in concrete.

In reinforced concrete, a long time trend is evident toward the use of high-strength materials both steel and concrete. In big housing projects, *high yield strength deformed bars* (HYSD bars) are in common use, having yield stress (0.2 per cent proof stress) equal to 415 N/mm^2 (4150 kg/cm^2) and permissible stress equal to 230 N/mm^2 (2300 kg/cm^2) for grade Fe 415. For grade Fe 500 HYSD bars, the yield stress is as high as 500 N/mm^2 while the permissible tensile stress is equal to 275 N/mm^2

TABLE 25.7.
PERMISSIBLE STRESSES IN STEEL REINFORCEMENT

Types of stress in steel reinforcement	*Permissible stresses*		
	(*a*) Mild steel bars conforming to grade I of IS 432 (Part I) 1966 or Deformed mild steel bars conforming to IS : 1139-1966	(*b*) Medium tensile steel conforming to IS : 432 (Part I) 1966 or Deformed medium tensile steel bars coforming to IS : 1139-1966	(*c*) High yield strength deformed bars conforming to IS : 1139 -1966 or IS : 1786-1979 (Grade Fe 415) (HYSD Bars)
1	2	3	4
1. Tension (σ_{st} or σ_{sv}) (*a*) upto and including 20 mm (*b*) over 20 mm	1400kg/cm^2 (140 N/mm 1300 kg/cm^2 (130 N/mm^2)	Half the guaranted yield stress subject to a maximum of 1900 kg/cm^2 (130 N./mm^2)	2300 kg/cm^2 (230 N/mm^2) 2300 kg/cm^2 (230 N/mm^2)
2. Compression in column bars (σ_{sc})	1300 kg/cm^2 (130 N/mm^2)	1300 kg/cm^2 (130 N/mm^2)	1900 kg/cm^2 (190 $N./mm^2$)
3. Compression in bars in a beam or slab when the compresive resistance of the concrete is taken into account.	The calculated compressive stress in the surrounding concrete multiplied by 1.5 times the modular ratio or σ_{sc} which is lower.		
4. Compression in bars in a beam or slab where the compressive resistance of the concrete is taken into account. (*a*) Upto and including 20 mm. (*b*) Over 20 mm.	1400 kg/cm^2 (140 N/mm^2) 1300 kg/cm^2 (130 N/mm^2)	Half the guaranteed yield stress subject to a maximum of 1900 kg/cm^2 (1900 $N./mm^2$)	1900 kg/cm^2 (190 $N./mm^2$) 1900 kg/cm^2 (19 $N./mm^2$)

PROBLEMS

1. Explain in brief various types of cements used in construction.
2. Write a note on composition and specification of Portland cement.
3. Write a note on aggregates used for cement concrete.
4. Explain : (*a*) Water cement ratio.
   (*b*) Workability of concrete.
   (*c*) Slump test.
5. Describe in brief various methods of proportioning concrete.
6. Explain the methods of mixing, compacting and curing of concrete.
7. Differentiate between plain cement concrete and reinforced cement concrete. Why is reinforcement necessary ?

26

Form Work

26.1. INTRODUCTION

The *form work* or *shuttering* is a temporary ancillary construction used as a mould for the structure, in which concrete is placed and in which it hardens and matures. The construction of form work involves considerable expenditure of time and material. The cost of form work may be upto 20 to 25% of the cost of structure in building work, and even higher in bridges. In order to reduce this expenditure, it is necessary to design economical types of formwork and to mechanize its construction. When the concrete has reached a certain required strength, the form is no longer needed and is removed. The operation of removing the form work is commonly known as *stripping*. When stripping takes place, the components of formwork are removed and then reused for the forms of another part of the structure. Such forms, whose components can be reused several times are known as *panel forms*. In contrast to this are *stationary forms* which are made for individual non-standard members and structures, which have no repeatable elements, and also for structural members, the form of work which cannot be stripped.

Forms are classified as wooden, plywood, steel, combined wood-steel, reinforced concrete and plain concrete. Timber is the most common material used for form work. The disadvantage of wooden form work is the possibility of warping, swelling and shrinkage of the timber. However, those defects can be overcome by applying to the shuttering water impermeable coatings. This coating also prevents the shuttering from adhering to concrete and hence makes the stripping easier. Steel shuttering is used for major work where every thing is mechanised. Steel form work has many advantages, such as follows : (*i*) it can be put to high number of uses, (*ii*) it provides ease of stripping, (*iii*) it ensures an even and smooth concrete surface, (iv) it possesses greater rigidity, (v) it is not liable to shrinkage or distortion. However, steel form work is comparatively more costly.

26.2. REQUIREMENTS

A good form work should satisfy the following requirements:

(*i*) The material of the form work should be cheap and it should be suitable for re-use several times.

(*ii*) It should be practically water proof so that it does not absorb water from concrete. Also, its shrinkage and swelling should be minimum.

(*iii*) It should be strong enough to withstand all loads coming on it, such a dead load of concrete and live load during its pouring, compaction and curing.

(*iv*) It should be stiff enough so that deflection is minimum.

(*v*) It should be as be light as possible.

(*vi*) The surface of the form work should be smooth, and it should afford easy stripping.

(*vii*) All joints of the form work should be stiff so that lateral deformation under loads is minimum. Also, these joints should be leak proof.

(*viii*) The form work should rest on non-yielding supports.

26.3. INDIAN STANDARD ON FORM WORK (IS : 456-1964)

1. ***General.*** The formwork shall conform to the shape, lines and dimensions, as shown on the plans and be so constructed as to remain sufficiently rigid during the placing and compacting of the concrete, and shall be sufficiently tight to prevent loss of liquid from the concrete.

2. ***Cleaning and treatment of forms.*** All rubbish, particularly chippings, shavings and sawdust, shall be removed from the interior of the forms before the concrete is placed and the form work in contact with the concrete shall be cleaned and thoroughly wetted or treated with an approved composition. Care shall be taken that such approved composition is kept out of contact with the reinforcement.

3. ***Stripping time.*** In no circumstances forms shall be struck until the concrete reaches a strength of at least twice the stress to which the concrete may be subjected at the time of striking.

The strength referred to shall be that of concrete using the same cement and aggregate, with the same proportions, and cured under condition of temperature and moisture similar to those existing on the work. Where possible, the form work should be left longer, as it would assist the curing.

In normal circumstances (generally where temperatures are above 20°C), and where ordinary cement is used, forms may be struck after expiry of following periods :

(*a*)	Walls, columns and vertical sides of beams	24 to 48 hours as may be decided by the engineer-in-charge.
(*b*)	Slabs (props left under)	3 days.
(*c*)	Beam soffits (props left under)	7 days.
(*d*)	Removal of props to slabs :	
	(*i*) Spanning upto 4.5 m	7 days.
	(*ii*) Spanning over 4.5 m	14 days.
(*e*)	Removal of props to beams and arches :	
	(*i*) Spanning upto 6 m	14 days
	(*ii*) Spanning over 6 m	21 days.

For rapid hardening cement 3/7 of the above period will be sufficient in all cases except vertical sides of slabs, beams and columns which should be retained for 24 hours.

Note. The number of props, their sizes and disposition, shall be such as to be able to safely carry the full load of the slabs, beam or arch as the case may be.

4. *Procedure when removing the form work.* All form work shall be removed without such shock or vibration as would damage the reinforced concrete. Before the soffit and struts are removed, the concrete surface shall be exposed, where necessary in order to ascertain that the concrete has sufficiently hardened. Proper precautions shall be taken to allow for the decrease in the rate of hardening that occurs with all cements in, the cold water.

5. *Camber.* It is generally desirable to give forms an upward camber to ensure that the beams do not have a sag when they have taken up their deflection, but this should not be done unless allowed for in design calculations of the beams.

6. *Tolerances.* Form work shall be so constructed that the internal dimensions are within the permissible tolerance specified by the designer.

26.4. LOADS ON FORM WORK

The form work has to bear mainly the following loads apart from its own weight : (*i*) live load due to labour etc. (*ii*) dead weight of wet concrete, (*iii*) hydrostatic pressure of the fluid concrete acting against the vertical or inclined faces of form, and (*iv*) impact due to pouring concrete. The temporary live loads of workmen and equipment, including the impact, may be taken equal to 370 kg/m^2 for the design of planks and joints in bending and shear. The hydrostatic pressure due to fluidity of concrete in the initial stages of pouring depends upon several factors such as, quantity of water in concrete, size of aggregates, rate of pouring and temperature. The hydrostatic pressure is maximum during pouring, and then decreases as concrete sets. Therefore, the *main* factor influencing this pressure

is the depth of concrete poured before the concrete sets. The setting time may be taken between $\frac{3}{4}$ to 1 hour. Hence while computing the pressure, only the height of concrete poured in $\frac{3}{4}$ to 1 hour need only be taken into account. For heights of concrete upto 1.5 m, the equivalent fluid weight of concrete may be taken as 2300 kg/m^3. For higher heights, this equivalent fluid weight is reduced. When the height of concrete in one pour is 6 m, the equivalent fluid weight may be taken as only 1200 kg/m^3. For intermediate heights between 1.5 to 6 m poured within the setting time of $\frac{3}{4}$ to 1 hour linear interpolation of unit weight between 2300 to 1200 kg/m^3 may be done.

Table 26.1 gives the safe values of stresses etc. for some common types of soft wood used for form work. The maximum permissible deflection of sheathing and joists etc. should not exceed 2.5 mm.

TABLE 26.1. PERMISSIBLE STRESSES IN TIMBER

Property	*Type of timber*			
	Fir	Deodar	Kail	Chir
Density (kg/m^3)	450	545	515	575
Modulus of elasticty E (tonne/cm^2)	94	95	68	98
Permissible stress in bending and tension (kg/cm^2) :				
(i) Inside	78	102	66	84
(ii) Outside	66	88	56	70
(iii) Wet	56	70	50	60
Permissible stresses in shear (kg/cm^2)				
(i) Horizontal	6	7	6	6
(ii) Along grain	8	10	8	9
Permissible compressive stress : parallel (kg/cm^2) :				
(i) Inside	60	78	52	64
(ii) Outside	52	70	46	56
(iii) Wet	42	56	38	46
Permissible compressive stress perpendicular (kg/cm^2) :				
(i) Inside	16	26	17	22
(ii) Outside	12	21	13	17
(iii) Wet	10	17	10	14

26.5. SHUTTERING FOR COLUMNS

Shuttering for a column is probably the simplest. It consists of the following main components : (*i*) sheeting all round the column

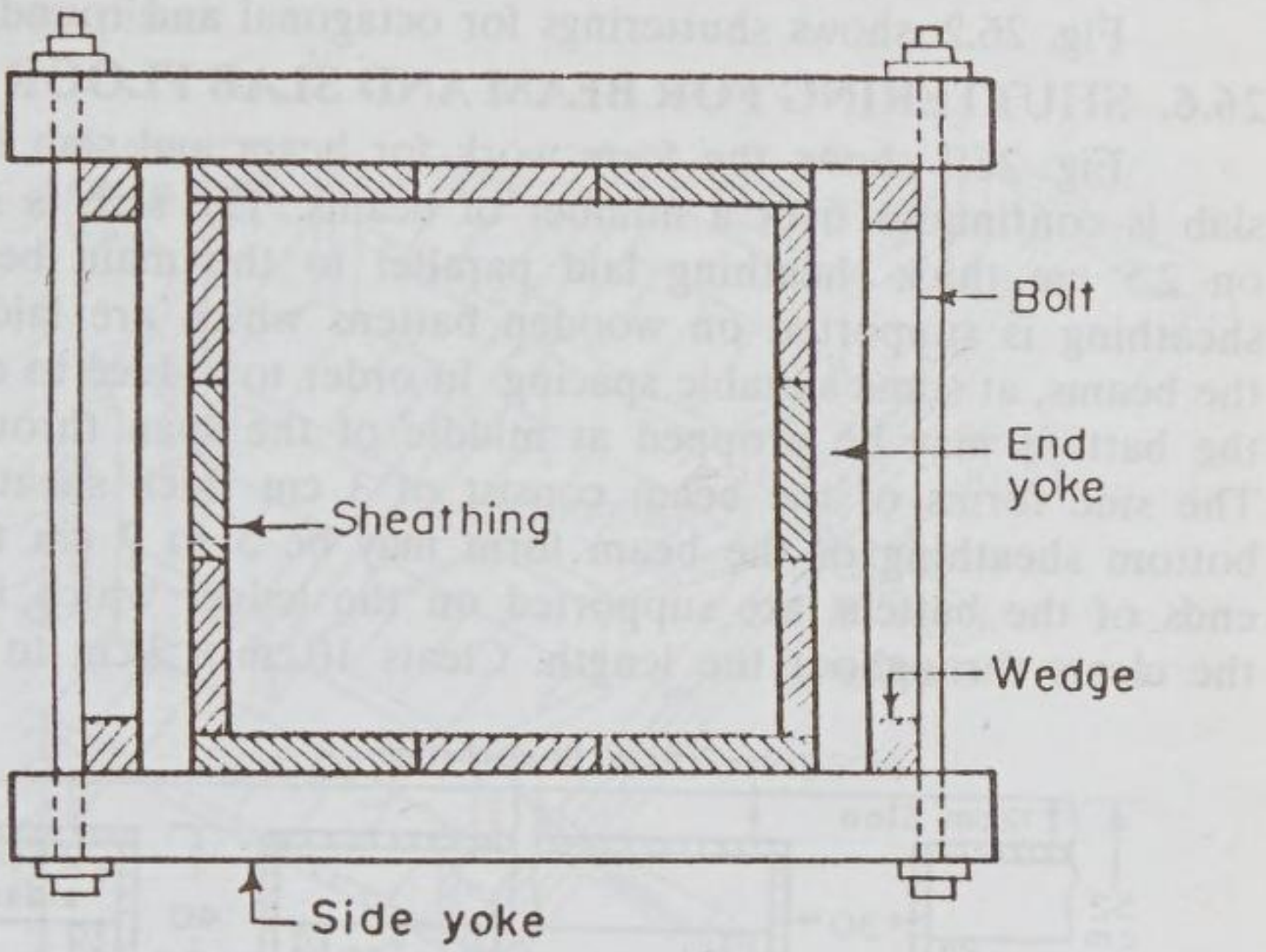
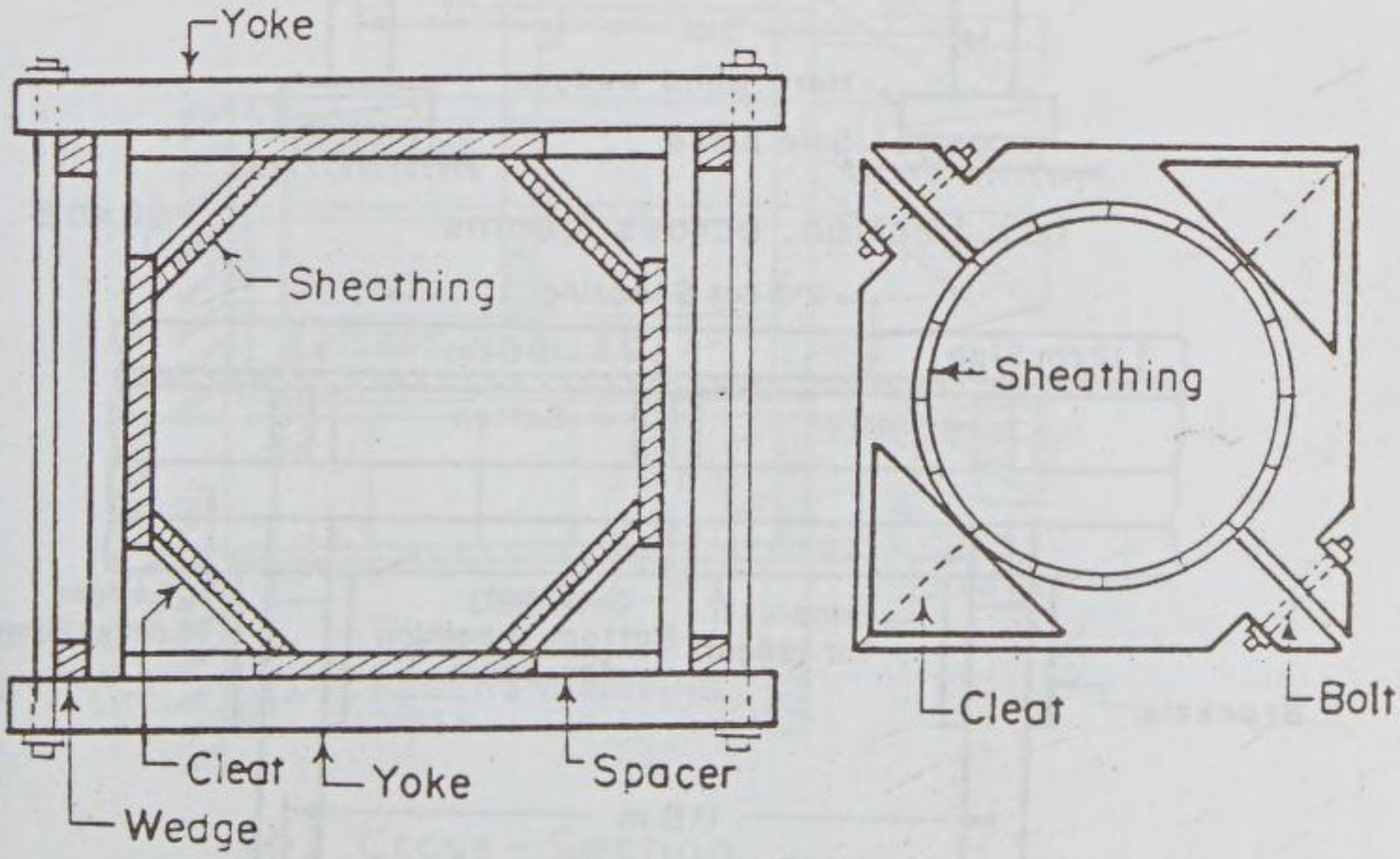


FIG. 26.1. FORM WORK FOR SQUARE OR RECTANGULAR COLUMN.

periphery, (*ii*) side yokes and end yokes, (*iii*) wedges, and (*iv*) bolts with washers. Fig. 26.1 shows the form work for a square column. The side yokes and end yokes consist of two numbers each, and are suitably spaced along the height of the column. The two-side yokes are comparatively of heavier section, and are connected together



(a) Octagonal Column (b) Round Column

FIG. 26.2

by two long bolts of 16 mm dia. Four wedges, one at each corner, are inserted between the bolts and the end yokes. The sheathing is nailed to the yokes.

Fig. 26.2. shows shutterings for octagonal and round columns.

26.6. SHUTTERING FOR BEAM AND SLAB FLOOR

Fig. 26.3 shows the form work for beam and slab floor. The slab is continuous over a number of beams. The slab is supported on 2.5 cm thick sheathing laid parallel to the main beams. The sheathing is supported on wooden battens which are laid between the beams, at some suitable spacing. In order to reduce to deflection, the battens may be propped at middle of the span through joists. The side forms of the beam consist of 3 cm thick sheathing. The bottom sheathing of the beam form may be 5 to 7 cm thick. The ends of the battens are supported on the ledger which is fixed to the cleats throughout the length. Cleats 10 cm × 2 cm to 3 cm are

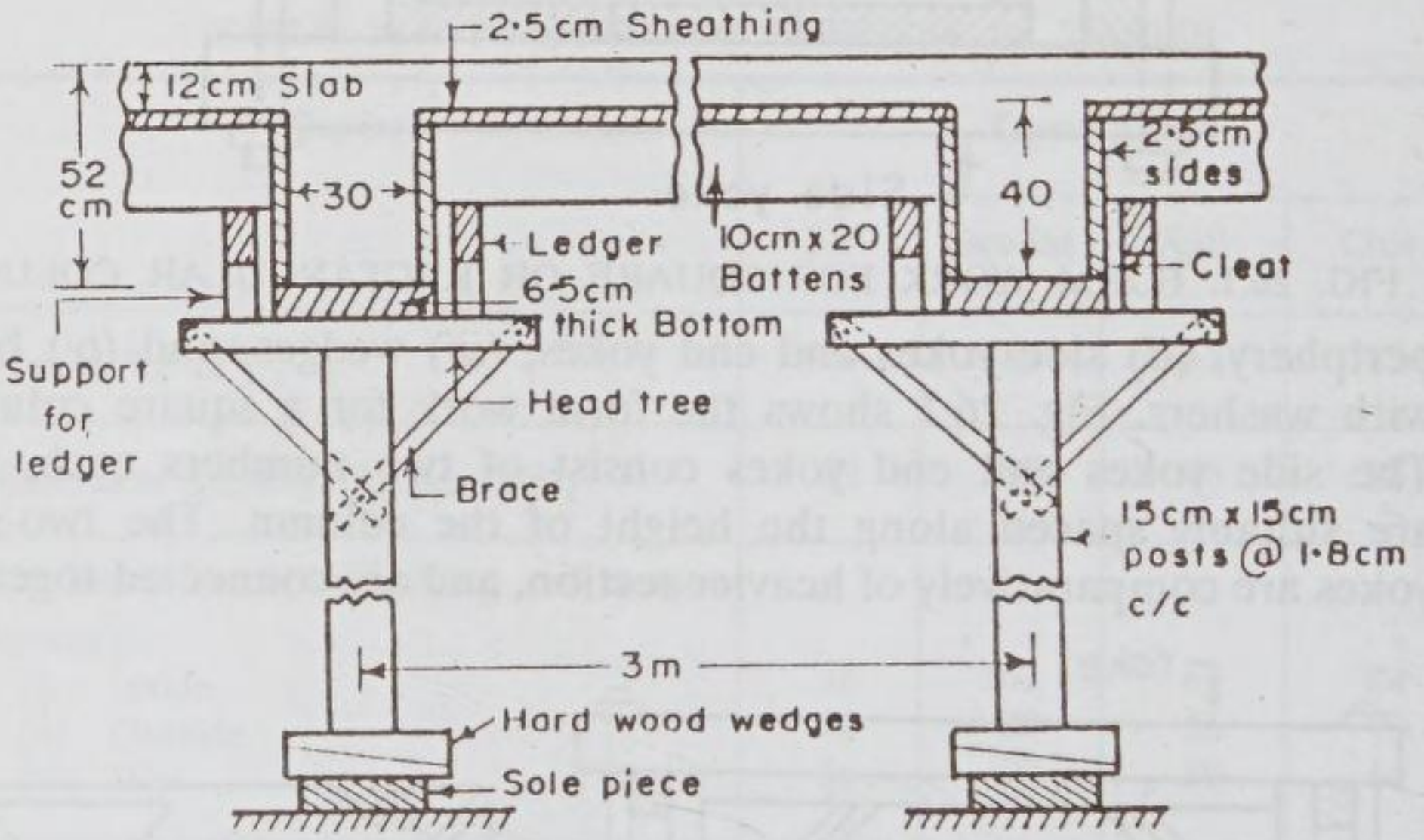


FIG. 26.3.

fixed to the side forms at the same spacing as that of battens, so that battens may be fixed to them. The beam form is supported on a head tree. The shore or post is connected to head tree through cleats. At the bottom of share, two wedges of hard wood are provided over a sole piece.

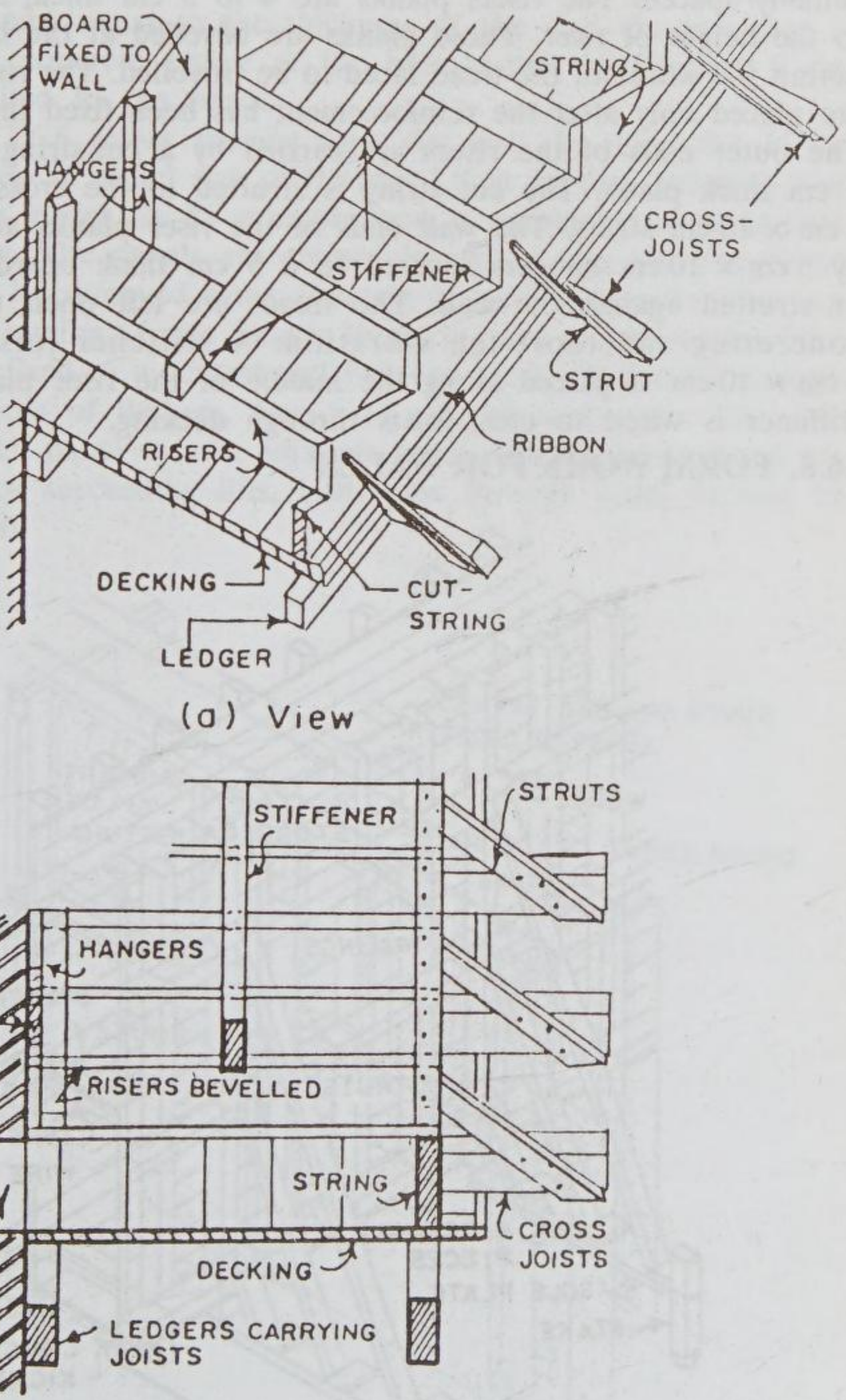


FIG. 26.4. FORM WORK FOR STAIRS

26.7. FORM WORK FOR STAIRS

Fig. 26.4 shows the form work for a stairs. The sheathing or decking for the deck slabs is carried on cross-joists which are in turn supported on raking ledgers. The ledger is generally of 7.5 cm × 10 cm size. The cross-joists may be of 5 cm × 10 cm size, suitably spaced. The risers planks are 4 to 5 cm thick, and equal to the height of riser. These planks are bevelled at the bottom to permit the whole of the tread faced to be trovelled. The riser planks are placed only after the reinforcement has been fixed in position. The outer ends of the risers are carried by a cut-string made of 5 cm thick plank. The cut string is strutted to the cross-joists by 5 cm × 10 cm struts. The wall ends of the riser planks are carried by 5 cm × 10 cm hangers secured to a 5 cm thick board fixed to or strutted against the wall. The treads are left open to permit concreting and thorough vibration. A stiffener joist of size 5 cm × 10 cm is placed along the middle of the riser planks. The stiffener is wired to cross-joists through decking.

26.8. FORM WORK FOR WALLS

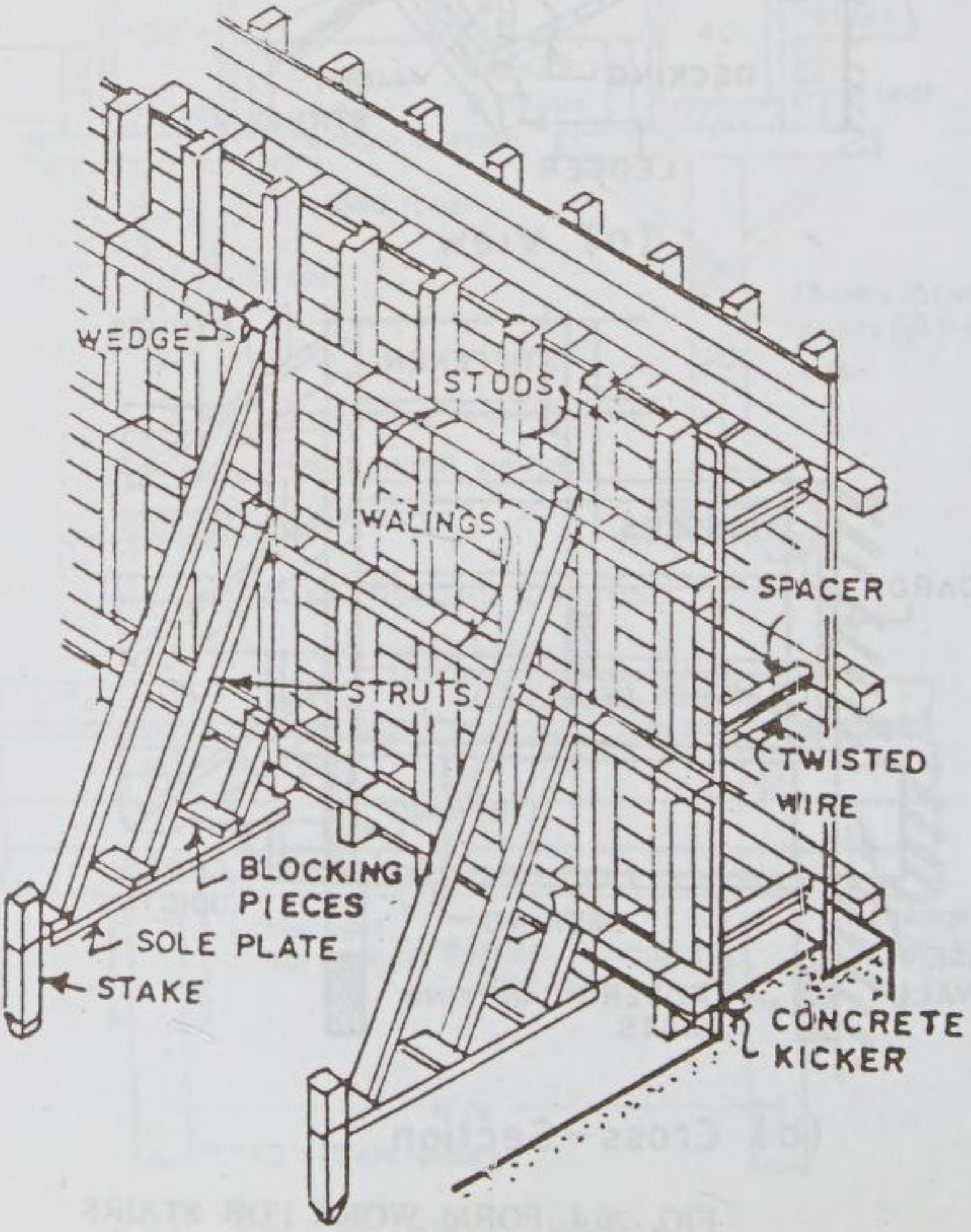


FIG. 26.5. FIXED FORM FOR WALL.

Fig. 26.5 shows fixed form for walls. The boarding may be 4 to 5 cm thick for walls upto 3 to 4 m high. The boards are fixed to 5 cm × 10 cm posts, known as studs or soldiers, spaced at about 0.8 m apart. Horizontal walings of size 7.5 cm × 10 cm are fixed to the posts at suitable interval. The whole assembly is then strutted as shown, using 7.5 cm 10 cm struts. The two shutters are kept apart equal to the thickness of the wall, by providing a 5 cm high concrete kicker at the bottom and by 2.5 cm × 5 cm spacers nailed to the posts.

Fig. 26.6 shows moving form for wall. In these the forms are made up in panel size of 0.6 m × 1.8 m so that handling and stripping is easier. A 15 mm plywood is commonly used instead of boarding. The panels are erected in such a way that the lower panels can be removed when concrete is hard and used higher up the wall. Framing of size 5 cm × 10 cm is used to ply shutter. The panels are fixed to a central and two end studs. Each stud consists of two pieces of timbers, 5 cm × 15 cm, blocked apart. The end strut of each panel secures adjacent panel. Boards are reversed for Ist lift ; for succeeding lifts, bolts pass through holes formed to previous lift.

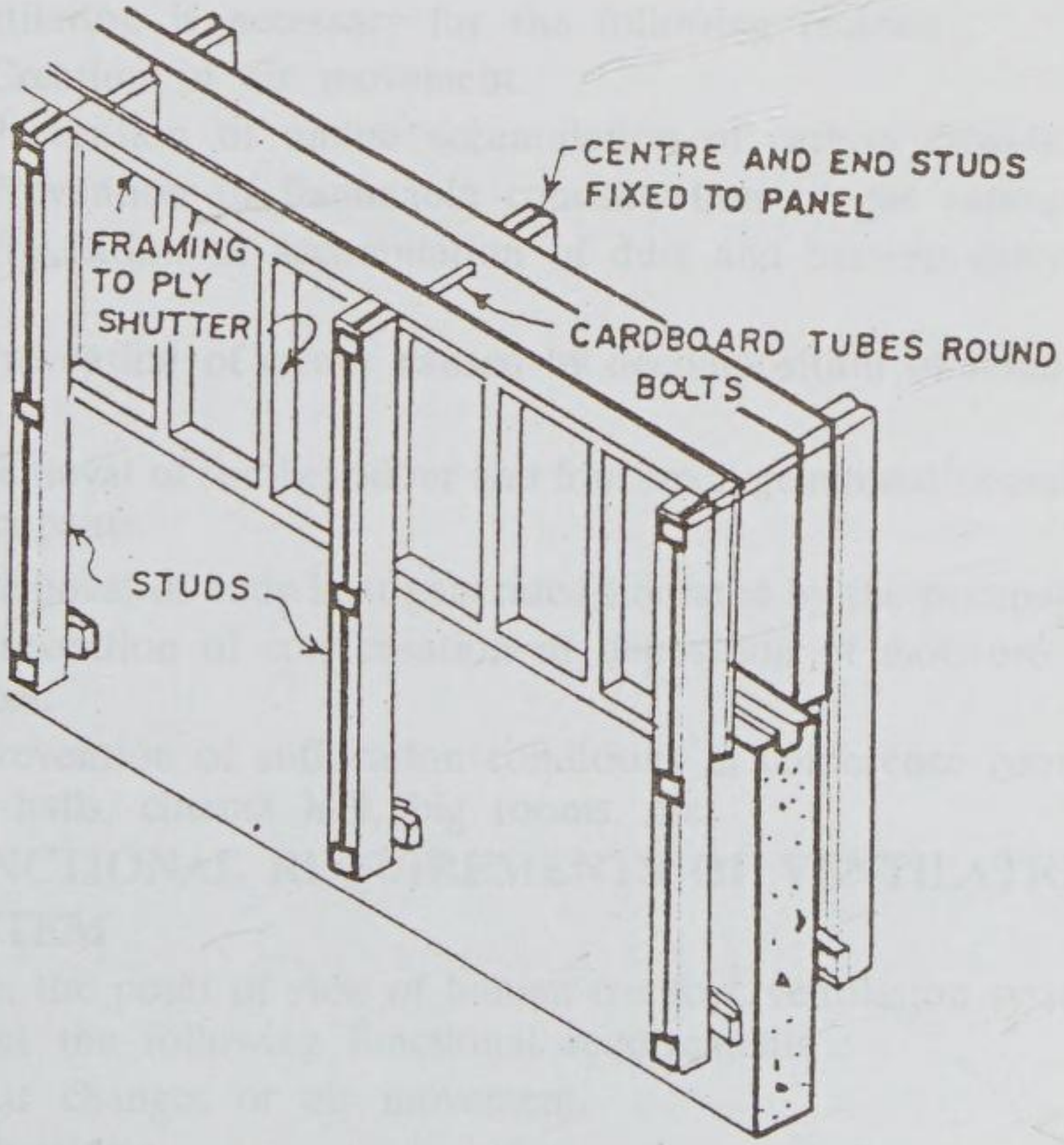


FIG. 26.6. MOVING WALL FORM.

For rapid construction of a constant thickness wall, continuously rising form, commonly known as *sliding shutter* is used. The shutter may rise at the rate of 15 to 30 cm per hour depending upon the rate of hardening of concrete. Either a hydraulic jack or a manually operated screw jack may be used for raising the form.

PROBLEMS

1. Draw a typical sketch for the form works of : (*a*) rectangular column, (*b*) Octagonal column.

2. Describe Indian Standard specifications for (*a*) cleaning and treatment of forms (*b*) Shipping time of form work.

3. Draw typical sketches of form work for a beam slab floor.

4. Describe how do you provide form work for a stair supported on wall on one side and stringer beam on the other side.

5. Explain with the help of sketches, timber form work for a 3 m high concrete wall.

27

Ventilation and Air-Conditioning

27.1. VENTILATION : DEFINITION AND NECESSITY

Ventilation may be defined as supply of fresh outside air into an enclosed space or the removal of inside air from the enclosed space. In other words, ventilation is the removal of all vitiated air from a building and its replacement with fresh air. Ventilation may be achieved either by natural or by artificial (or mechanical) means.

Ventilation is necessary for the following reasons :

1. Creation of air movement.
2. Prevention of undue accumulation of carbon dioxide.
3. Prevention of flammable concentration of gas vapour.
4. Prevention of accumulation of dust and bacteria-carrying particles.
5. Prevention of odour caused by decomposition of building material.
6. Removal of smoke, odour and foul smell generated/liberated by the occupants.
7. Removal of body heat generated/liberated by the occupants.
8. Prevention of condensation or deposition of moisture on wall surfaces.
9. Prevention of suffocation conditions in conference rooms, committee halls, cinema hall, big rooms, etc.

27.2. FUNCTIONAL REQUIREMENTS OF VENTILATION SYSTEM

From the point of view of human comfort, ventilation system should meet the following functional requirements :

1. Air changes or air movement.
2. Humidity.

3. Quality of air.
4. Temperature.

1. Air changes (or air movement) and rate of supply of fresh air

In an enclosed space, where people are working or living, air has to be moved or changed to cause proper ventilation. The minimum rate of air change is one per hour, while the maximum rate of air change is sixty per hour. Air change per hour is the volume of outside air allowed in the room or enclosed space per hour compared to the volume of the room. If the rate of air change is less than one per hour, there will be no ventilation, while if the rate of air change is more than sixty per hour it will cause discomfort to the occupants because of high velocity of air. Cross-ventilation is provided to increase the rate of air movement in a naturally ventilated building while fans etc. are used in case of mechanically ventilated buildings.

Since the amount of fresh air required to maintain the carbon dioxide concentration of air within safe limits and to provide sufficient oxygen content to air for respiration is very small and since the rate of ventilation to maintain satisfactory thermal environment for a region varies from season to season, the minimum standards of ventilation are based on control of body odour or the removal of products of combustion depending on the requirements of each case.

The volume of fresh air required for the removal of body odour is influenced by the air space per person–the volume decreases as the air space per person increases. A rough guidance can be taken from the following table :

Air space per person (m^3)	*Fresh air supply per person* (m^3/h)
5.5	28.5
8.5	20.5
11.0 and upwards	17.0

Indian Standard, IS : 3362-1965 recommends the following values for residential buildings :

(*i*) *Living rooms and bed rooms.* In the case of living rooms and bedrooms, minimum of three air changes per hour should be provided.

(*ii*) *Kitchens.* Large quantity of air are needed to remove the steam, heat, smell and fumes generated in cooking and to prevent excessive rise of temperature and humidity. However, for the requirement of kitchen in which cooking is done for a family of not more than five persons, minimum rate of ventilation of about three air changes per hour should be provided.

(*iii*) *Bath rooms and water closets.* Considerable ventilation

of bathrooms and water closets is desirable after use, and the equivalent of three air changes per hour should be provided.

(*iv*) *Passages.* The period of occupation of passages lobbies and the like is very short, and as such no special consideration in designing their ventilation system.

Indian Standard has not made any recommendations for ventilation standards of public buildings. However, guidance may be taken from Table 27.1.

TABLE 27.1. RECOMMENDATIONS FOR VENTILATION IN PUBLIC BUILDINGS

Type of building	*Minimum rate of fresh air in the building*
1. Assembly halls, canteens, shops, restaurants etc.	30 m^3 Person per hour.
2. Factories and workshops	
(*i*) work rooms	25 m^3 per person per hour.
(*ii*) lavatories	2 air changes per hour
3. Hospitals	
(*i*) operation theatres etc.	10 air changes per hour.
(*ii*) wards	3 air changes per hour.
4. Schools	
(*i*) rooms ; space provided between 8 m^3 per person to 5 m^3 per person.	20 to 30 m^3 per person per hour respectively.
(*ii*) Corridors, lavatories and w.c.'s.	2 air changes per hour.
5. Offices	
(*i*) Office rooms with space from 5 m^3 per person to 11 m^3 per person.	30 m^3 per person per hour to 17 m^3 per person per hour.
(*ii*) Lavatories and w.c.'s	2 air changes per hour.

2. Humidity. Air contains certain amount of water vapour in it. *Relative humidity* is defined as the ratio of amount of water vapour present in the air to the amount of water vapour if the air were saturated at the same temperature. Thus, the relative humidity of saturated air is 100 percent. Relative humidity within the range of 33 to 70 per cent at the working of 21°C, is considered to be desirable. For higher temperatures, low humidity and greater air movements are necessary for removing greater portion of heat from the body.

3. Quality of air. The ventilating air should be free from impurities, odours, organic matter and inorganic dust. It should also be free from unhealthy fumes of gases, such as carbon monoxide, carbon dioxide, sulphur dioxide etc. The ventilating air should not come from the vicinity of chimneys, kitchens, latrines, urinals, stables etc. Air containing less than 0.5 mg of suspended impurity per m^3 and less than 0.5 part per million of sulphur dioxide is considered

to be clean, and does not require further treatment. Air within the room containing 0.06 percent of CO_2 may be considered vitiated, but with 0.09 or 0.1 percent, it becomes stuffy and unbearable. Hence the air in habitable rooms should never contain more than 0.06 percent of CO_2. The air should be kept in this condition by proper ventilation. *Pure air in buildings is necessary for the substenance and improvement of health, for the perfect combustion of fuel and for the preservation of materials of which the building is constructed.*

4. Effective temperature. It is desirable that the incoming ventilating air should be cool in summer and warm in winter, before it enters the room. The general temperature difference between inside and outside is kept not more than 8°C. With regard to human comfort the term *effective temperature* is more useful. It is an *index* which combines into a single value, the effect of air movement, humidity and temperature. Effective temperature indicates the temperature of air at which a person will experience sensation of same degree of cold or warmth as in quite air fully saturated (*i.e.* 100% humidity) at the same temperature. In other words, it is the effective temperature which is more important than the actual temperature itself. If two rooms have the same effective temperature, a person leaving one room and entering the other will not experience any change of temperature though the actual temperatures in the two rooms may be different. The value of effective temperature, from human comfort point of view, depends upon the type of activity, geographical conditions, age of occupants, amount of heat loss from the body etc. The common values of effective temperatures in winter and summer are 20°C and 22° C respectively.

27.3. SYSTEMS OF VENTILATION

Systems of ventilation may be divided into two categories

(*i*) Natural ventilation.

(*ii*) Mechanical ventilation or artificial ventilation.

Natural ventilation is the one in which ventilation is effected by the elaborated use of doors, windows, ventilators and skylights. It is usually considered suitable for residential buildings and small houses. However, it is not useful for big buildings, offices, conference halls, auditoriums, large factories etc. In natural ventilation, cross ventilation is normally relied to secure air movement. It is economical since no equipment is required for keeping the room ventilated.

Mechanical ventilation is the one in which some mechanical arrangements are made to increase the rate of air flow. The system is more usefull for large buildings, assembly halls, factories, theatres, etc. Though the system is more costly, it results in considerable efficiency of the persons using the building.

27.4. NATURAL VENTILATION

In this system, ventilation is effected by doors, windows, ventilators, skylights and other openings in the enclosed space. The rate of ventilation depends on two effects :

(*a*) Wind effect

(*b*) Stack effect.

(*a*) Ventilation due to wind effect

In this, the rate of ventilation depends upon the direction and velocity of wind outside and sizes and positions of openings. Such an effect is known as 'ventilations due to wind action'. When wind blows at right angles to one face of a building, pressure differences are created–positive pressure is produced on windward face and negative pressure (or suction) is produced on the leeward face. If the wind direction is at 45° to one of the faces, positive pressure will be produced on two windward faces and negative pressure on the two leeward faces. Fig. 27.1 shows the movement of wind through buildings.

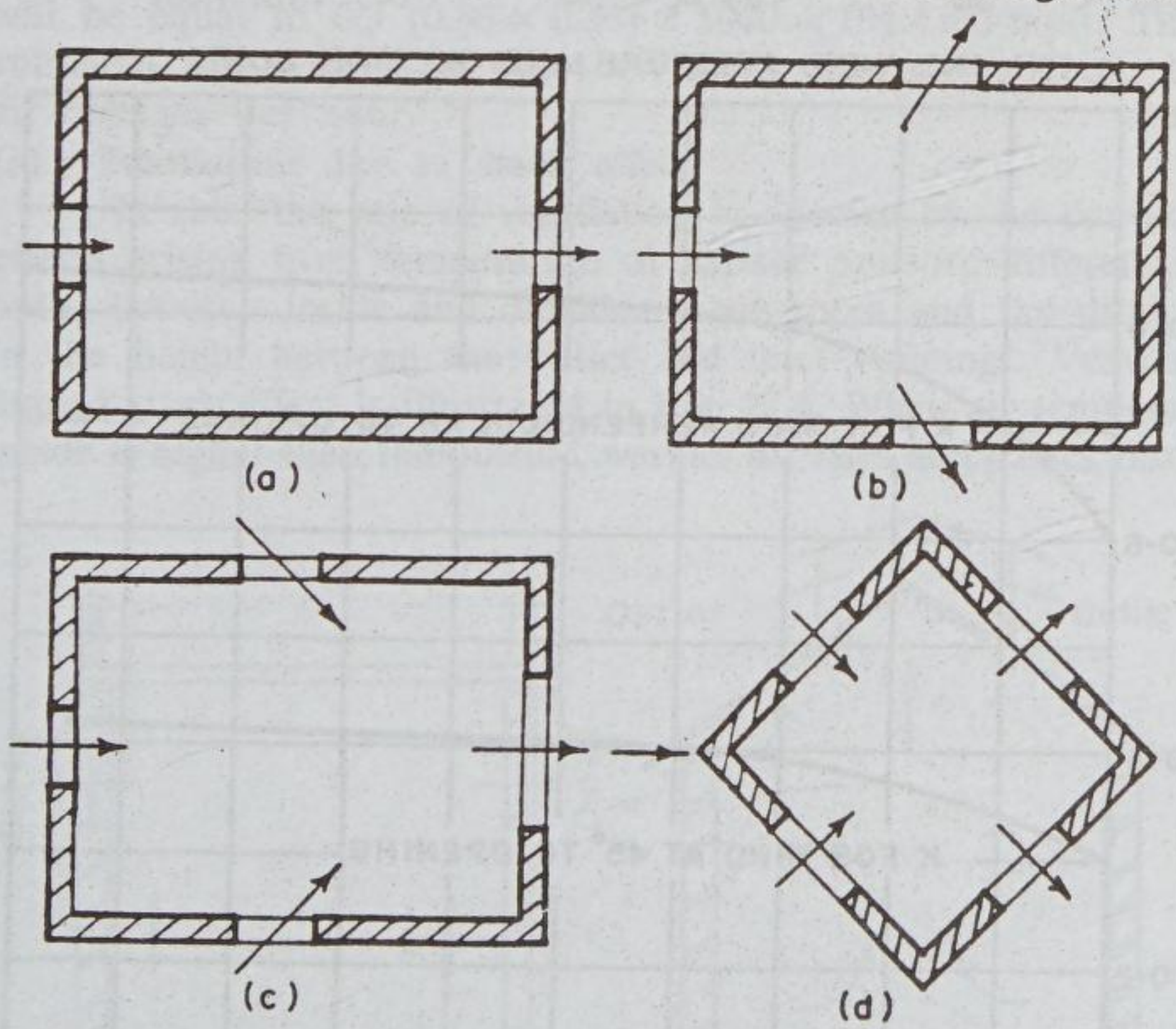


FIG. 27.1. MOVEMENT OF WIND THROUGH BUILDINGS.

In designing a system of natural ventilation, the aim should be to make effective use of wind forces. Since these are not constant, being dependent on the speed and direction of wind, it is obvious that the ventilation is likely to be variable in quantity. For design purposes, the wind may be assumed to come from any direction

within 45° of the direction of prevailing wind.

In the case of pitched roof, the pressure will depend upon the pitch of the roof, as shown in Fig. 27.2. It is seen that the roof pressures in general are *negative*, except on the windward side of the roof with shape greater than 30°. Wind will blow from windward side to the other side if there is an opening.

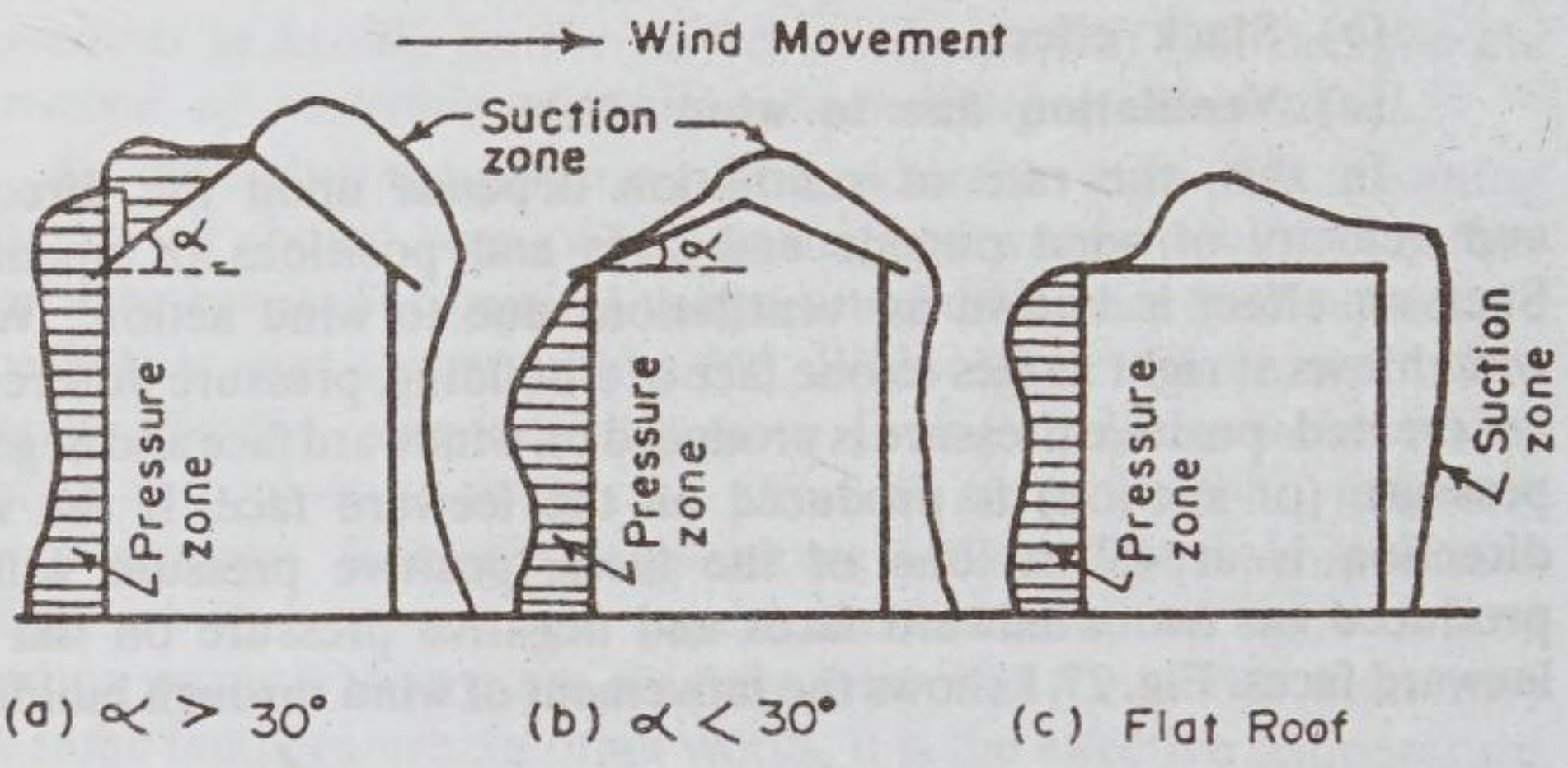


FIG. 27.2. WIND PRESSURE AND SUCTION ZONE.

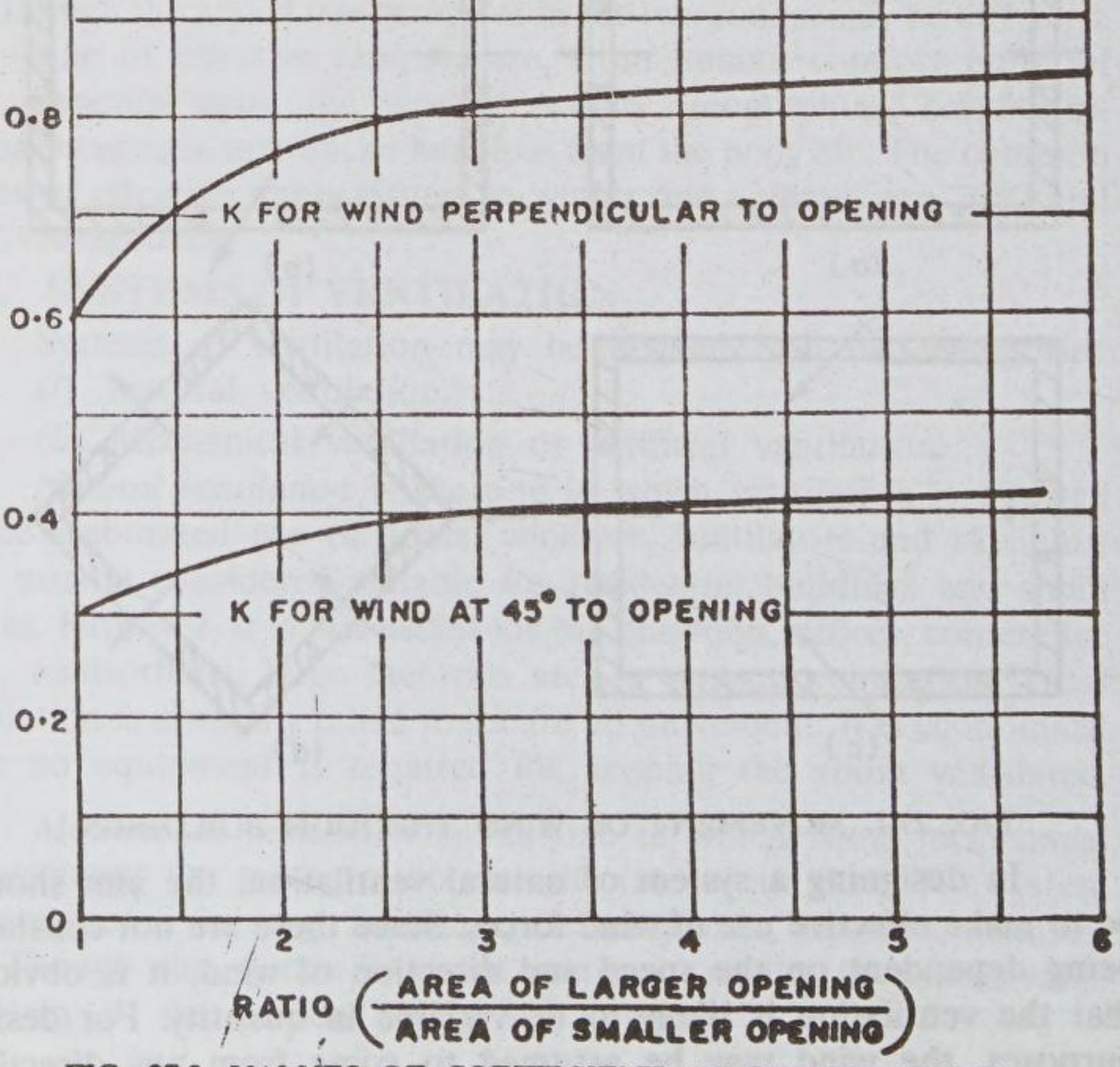


FIG. 27.3. VALUES OF COEFFICIENT OF EFFECTIVENESS K FOR FLOW THROUGH TWO OPENINGS.

Rate of air flow in wind effect

Considering the simple case of an isolated enclosure in which an opening is provided, in each of two opposite walls, the rate of air flow through an opening, due to wind blowing on the wall containing the opening is given by the expression

$$Q = KAV. \quad ...(27.1)$$

where Q = the rate of air flow, in m^3/h

K = coefficient of effectiveness

A = area of smaller opening, in m^2 and

V = wind speed in m/h.

The coefficient of effectiveness K depends upon the direction of the wind relative to the opening, and on the ratio between the areas of the two openings. It is a maximum when the wind blows directly on the opening and it increases with the relative size of the larger opening. Fig. 27.3 gives the values of K.

Thus, the flow through two square openings of size 0.36 m, with a wind of 5 km/hour blowing inclined at 45° to the opening will be equal to $0.3\ (0.36 \times 0.36) \times 5000 = 194.4\ m^3$/hour. This is sufficient for a room of $4 \times 4 \times 4$ m in size, giving about three air changes per hour.

(*b*) Ventilation due to stack effect

In this, the rate of ventilation is affected by the convection effects arising from temperature or vapour pressure difference (or both) between inside and outside of the room and the difference in the height between the outlet and inlet openings. Ventilation due to stack effect is illustrated in Fig. 27.4. When air temperature inside is higher than the outside, warmer air rises and passes through

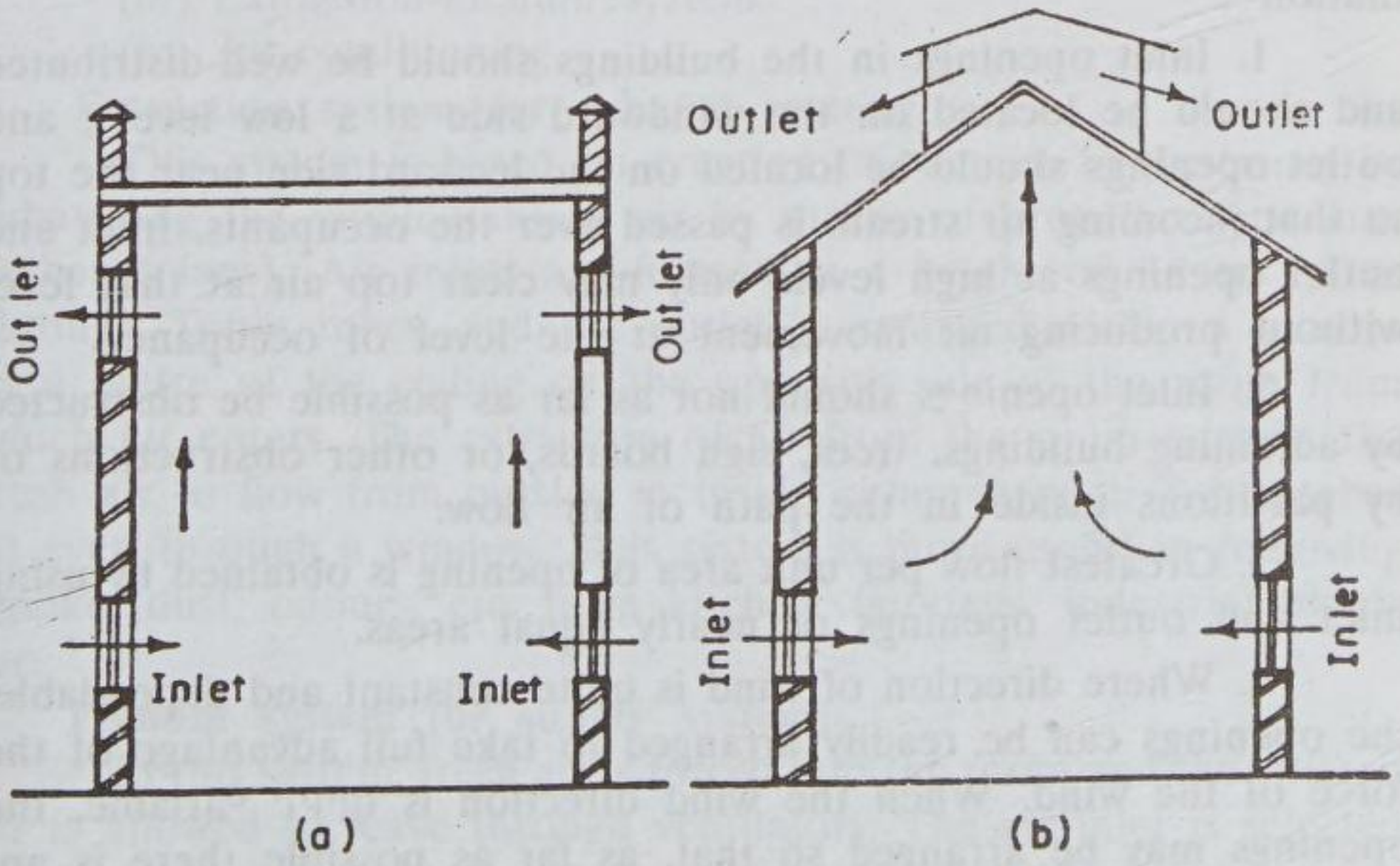


FIG. 27.4. VENTILATION DUE TO STACK EFFECT.

opening located in the upper part of the room, whereas incoming cool air enters from the lower openings.

The rate of air flow in stack effect :

The rate of air flow arising from temperature difference between outside and inside is given by

$$Q = 640\, C\, .\, A\, .\, \sqrt{h(t_i - t_o)} \qquad \text{....(27.2)}$$

where Q = rate of air flow in m^3/h

C = coefficient of effectiveness
= 0.65 for general conditions
= 0.50 for unfavourable conditions

A = free area of inlet opening

h = vertical height difference between inlet and outlet in m

t_i = average temperature of inside air, in °C

t_o = average temperature of outside air, in °C.

Ventilation due to both the effects

When both wind and stack pressures are acting, it is proper to calculate each pressure acting independently under conditions ideal to it and then apply a percentage. However, ventilation in residential buildings due to stack pressure both in hot-arid region and in hot-humid region appears to be insignificant and at any rate may be neglected, as when both wind pressure and stack pressure are acting, the wind pressure effect may be assumed to be predominant.

General rules of natural ventilation. Indian Standard Code IS : 3362-1965 lays down the following general rules of natural ventilation :

1. Inlet openings in the buildings should be well-distributed and should be located on the windward side at a low level ; and outlet openings should be located on the leeward side near the top so that incoming air stream is passed over the occupants. Inlet and outlet openings at high levels only may clear top air at that level without producing air movement at the level of occupancy.

2. Inlet openings should not as far as possible be obstructed by adjoining buildings, trees, sign boards, or other obstructions or by partitions inside in the path of air flow.

3. Greatest flow per unit area of opening is obtained by using inlet and outlet openings of nearly equal areas.

4. Where direction of wind is quite constant and dependable, the openings can be readily arranged to take full advantage of the force of the wind. When the wind direction is quite variable, the openings may be arranged so that, as far as possible there is approximately equal area on all sides. Thus no matter what the wind

direction is there are always some openings directly exposed to wind pressure and others to air suction and effective movement through building is assured.

5. Natural ventilation occurs when the air inside a building is at a different temperature than the air outside. Thus in a heated building and in an ordinary building during summer nights and during pre-monsoon period when the inside temperature is higher than outside, cool outside air will tend to enter through openings at low level and warm air will tend to leave through openings at high level. It would, therefore, be advantageous to provide ventilators as close to the ceiling as possible. Ventilators can also be provided in roofs, as for example, cowl, vent pipe, covered roof and ridge vent.

6. Windows of living rooms, should either open directly to an open space or to an unobstructed facing on open space. In places where building sites are restricted, open space may have to be created in the buildings by providing adequate courtyards.

27.5. MECHANICAL (OR ARTIFICIAL) VENTILATION

Mechanical ventilation or artificial ventilation involve the use of some mechanical equipment for effective air circulation. It is provided in those circumstances where satisfactory standard of ventilation in respect of air quantity, quality or controlability cannot be obtained by natural means. This system is costly, but it results in considerable increase in the efficiency of persons under the command of the system. There are following systems of artificial ventilation :

(*i*) Extraction system.
(*ii*) Plenum system.
(*iii*) Extraction-Plenum system.
(*iv*) Air-conditioning.

1. Extraction system (or exhaust system)

This system is based on creation of vacuum in the room by exhausting the vitiated inside air by means of propeller type fans (exhaust fans). Air inlets are formed at a height of 1.2 to 1.8 m through Tobin tubes, and the outlet is arranged within a quarter of a metre of the ceiling on the opposite side of the room from which air enters. The extraction of air from the room permits the fresh air to flow from outside to inside either through Tobin tubes or even through a window. This system is more useful in removing smoke, dust, odours, etc. from kitchen, laterines, industrial plants etc.

2. Plenum system (or supply system)

In this system, fresh air is forced into the room and the vitiated air is allowed to leave through ventilators. The air inlet is selected on that side of the building where purest air is available. The incoming air which is mechanically forced into the room is passed through

a fine gauge screen or filter. A constant stream of water is kept flowing down the screen giving a fine mist of water through which the air is drawn by means of blower fan. Thus, all the mechanical impurities are removed from the air. In summer, this also results in *cooling* of air. At this point air may be further disinfected by the introduction of ozone from an ozonizing apparatus. In winter, the air may be forced through a battery of hot water tubes and be heated before being forced into the room. In the case of big hall or factories etc., the distribution of this air is done through properly formed sheet iron ducts with properly dimensioned branches. This ventilation system is costly, but is used for factories, conference halls, theatres, big offices, etc.

The ventilation by plenum process may be either downward or upward. In the downward ventilation, the incoming air is allowed to enter at the ceiling height and while mixing with the vitiated air during its downward journey, it is taken out through outlets situated at the floor level. In the upward system, fresh air enters at the floor level and moves out at the ceiling level.

3. Extraction-Plenum system

This is an extension of plenum system in which extraction fans are used for the exit of the vitiated air from the room. This system is adopted where the delivery of fresh air is either sluggish or where it is desired to discharge vitiated air containing obnoxious fumes as from kitchens, laterines, or various manufacturing processes, in specially isolated areas.

4. Air conditioning

This is the best system of artificial ventilation in which provision is made for filtration, heating or cooling, humidifying or dehumidifying etc, thus creating most comfortable working condition.

27.6. AIR CONDITIONING

Air-conditioning may be defined as the process of treating air so as to control simultaneously its temperature, humidity, purity and distribution to meet the requirements of the conditioned space. The various requirements of a conditioned space may be comfort and health of human beings, needs of certain industrial processes, efficient working of commercial premises etc.

Air-conditioning is resorted to for the following normal

(*i*) It helps in preserving or maintaining hetlth, comfort and convenience of occupants of residential building.

(*ii*) It helps in improving the quality of products in certain industrial processes such as artificial silk, cotton cloth. etc. In other cases of industries, it provides comfortable working conditions for the workers, resulting in the increase of the production.

(*iii*) It helps in marking the commercial premises, such as

shops banks, offices, restaurants etc., more active and efficient.

(*iv*) It provides more comfortable entertainment in theatres etc.

(*v*) In the case of air conditioned railway/roadways coaches, or air travel, journey becomes more comfortable.

Functional Classification

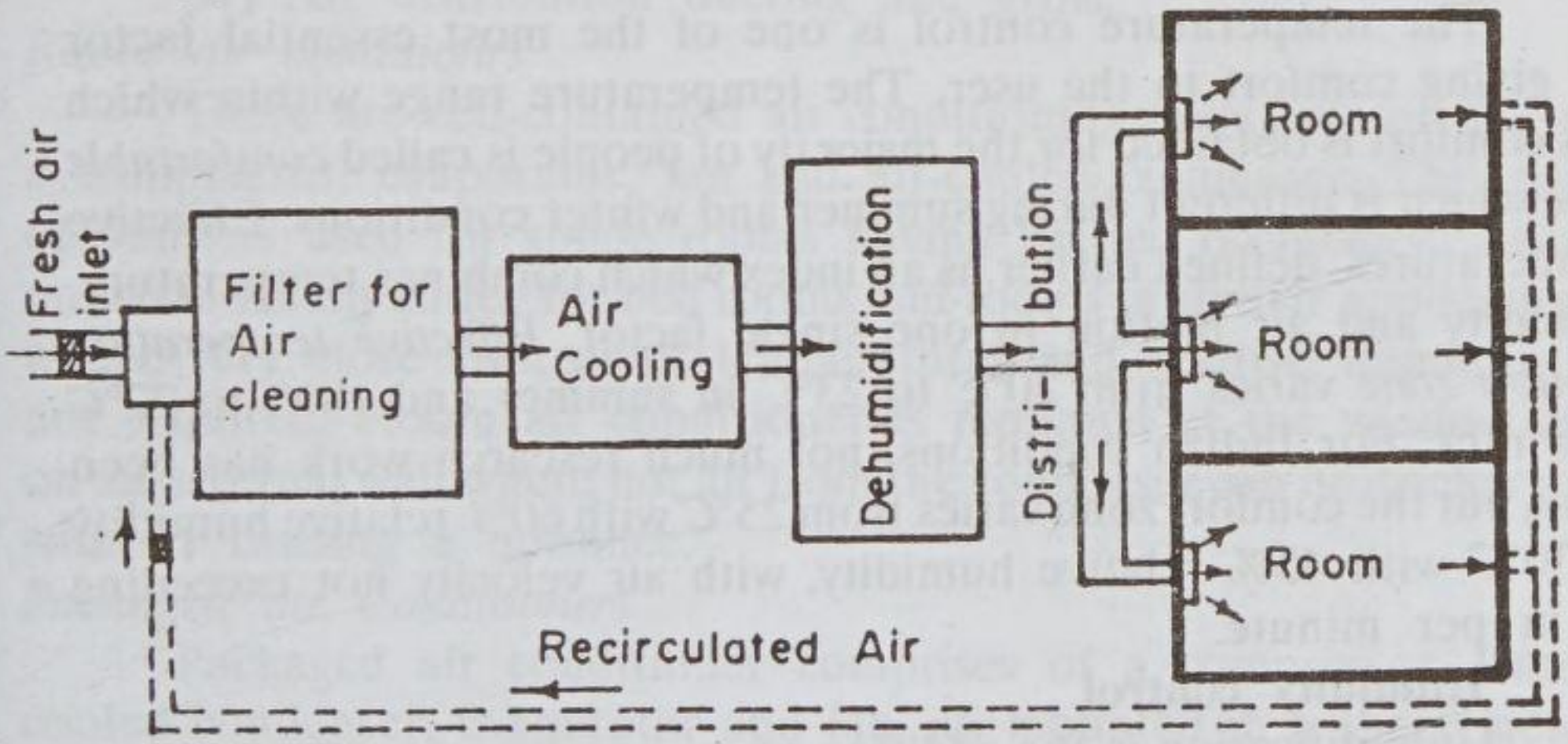
From functional point of view, air conditioning may be of two types :

(*i*) *Comfort air conditioning.* In this, the system aims at giving maximum human comfort to the occupants/users of the conditioned space.

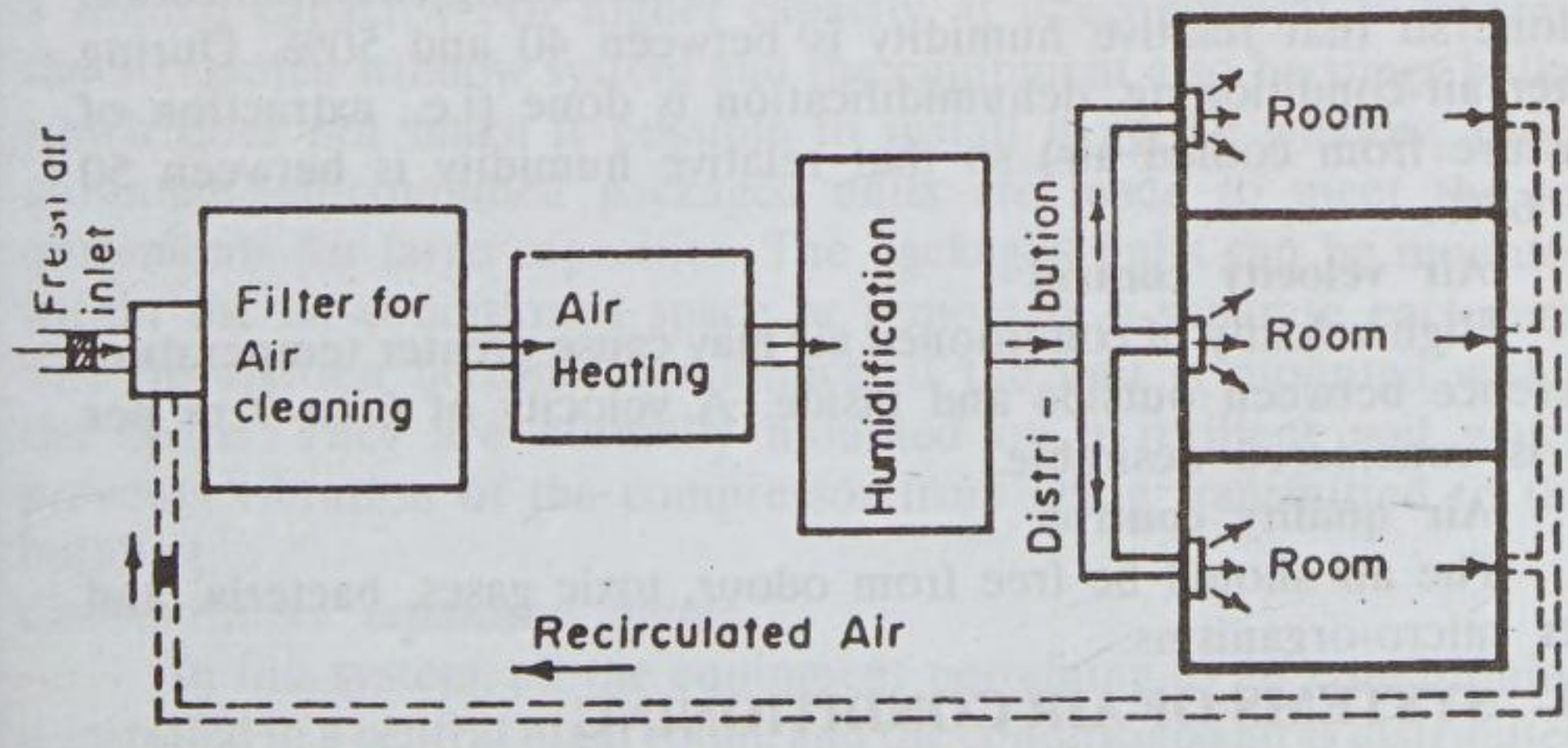
(*ii*) *Industrial air conditioning.* In tnis, the conditioning creates, controls and maintains such an environment inside the conditioned space, that would suit best to the needs of the industry.

Classification based on season/temperature

(*a*) *Summer air conditioning.* In summer, outside temperature



(a) Summer Air-conditioning



(b) Winter Air-conditioning

FIG. 27.5. CYCLES OF OPERATIONS IN AIR CONDITIONING.

is more, and hence cooling of air is required for greater comfort. The cycle of operations consists of (*i*) air cleaning, (*ii*) air cooling, (*iii*) dehumidification, and (*iv*) air distribution/circulation. This is shown diagrammatically in Fig. 27.5 (*a*).

(*b*) *Winter air conditioning.* In winter, outside temperature is low, and hence heating of air is required for comfort. The cycle of operations consists of (*i*) air cleaning, (*ii*) air heating, (*iii*) humidification, and (iv) air distribution/circulation, as shown in Fig. 27.5 (*b*).

(*c*) *Composite air conditioning.* In this, the same air conditioning is done throughout the year, irrespective of outside temperature.

27.7. ESSENTIALS OF COMFORT AIR CONDITIONING

Comfort air conditioning requires the proper control of the following; (*i*) temperature control, (*ii*) humidity control, (*iii*) air velocity control and (*iv*) air quality control.

1. Temperature control

The temperature control is one of the most essential factor for giving comfort to the user. The temperature range within which this comfort is obtained for the major ity of people is called *comfortable zone* which is different during summer and winter conditions. Effective temperatures, defined earlier, is an index which combines temperature, humidity and air motion in one single factor. *Effective temperature comfort zone* varies from 20°*C* to 23°*C* in summer and 18°*C* to 22°*C* in winter. For Indian conditions, not much research work has been done, but the comfort zone varies from 25°*C* with 60% relative humidity to 30°*C* with 45% relative humidity, with air velocity not exceeding 10 m per minute.

2. Humidity control

Humidification is the addition of moisture to the dry air which otherwise would cause great strain and irritation on the membranes of nasal passages. During summer air conditioning, humidification is done so that relative humidity is between 40 and 50%. During winter air-conditioning, dehumidification is done (i.e., extraction of moisture from cooled air) so that relative humidity is between 50 and 60%.

3. Air velocity control

High velocity of conditioned air may cause greater temperature difference between outside and inside. A velocity of 6 to 9 m per sec. is considered desirable.

4. Air quality control

The air should be free from odour, toxic gases, bacteria, and other micro-organisms.

27.8. SYSTEMS OF AIR CONDITIONING

All air conditioning systems can be broadly classified in two categories.

(*a*) Direct Expansion (DX) systems
(*b*) Chilled water (indirect) systems.

1. Direct Expansion System

It is the system where the refrigerant is utilised to cool the air directly. The common examples of this system are :

(*a*) Room air conditioner.
(*b*) Packaged units.
(*c*) Central direct expansion plants.

The main components of the direct expansion system are :

(*i*) Compressor (hermetic or open type).
(*ii*) Condenser (air cooled or water cooled).
(*iii*) Evaporater or cooling coil with fan.
(*iv*) Cooling tower or spray panel (in case of water-cooled condensor)
(*v*) Condenser water pump set.
(*vi*) Air distribution ducting and grills.

Room air conditionrs

There are self-contained air conditioning units, comprising of a compressor, evaporator, fan and air-cooled condenser. This type of plant is used for single rooms having limited occupancy. These units are ideally suited for bed rooms and rooms of similar application, where very close control of temperatures and relative humidity is not required. Room air conditioner is mounted at the window sill on an external wall where hot air from the condenser can be discharged without causing a nuisance.

Packaged air Conditioners

Packaged air conditioner comprises of a compressor, water cooled condenser, evaporator and fan, all mounted in a sheet metal cabinet. They are ideally suited for residences, shops, banks, offices and some industrial applications. Window units are available upto a limited capacity. For higher capacity, it is economical to operate the air-cooled window system and the equipment also becomes bulky, which does not make it possible to install it at the window. Floor mounted self-contained packaged units are made to meet the requirements for large capacities. The packaged units can be mounted within the air-conditioned space or remote in a separate enclosure. The installation becomes economical if the unit is mounted within the space. They are normally mounted on a resilient pad which prevents vibration of the compressor from being transmitted to the building.

Central direct expansion plants

In this system, all the equipment pertaining to air conditioning is installed in a central plant room, and the conditioned air is distributed to the rooms/halls/enclosures by ducts. The system is more useful

for factories, hotels, assembly halls, cinema halls and big residential buildings where it is uneconomical to install separate units for each room.

2. Chilled Water System

In this system, secondary medium, such as water is used to cool the air. The refrigerant first cools the water and then the water cools the air. The main components of this system are the same as direct expansion system but a chiller is the additional item. Following is the list of the components of the system :

(*i*) Compressor (reciprocating or centrifugal),
(*ii*) Condenser (mostly water-cooled).
(*iii*) Chiller (direct expansion or flooded type).
(*iv*) Air handling unit with chilled water coil and/or fan coil unit or chilled water air washers.
(*v*) Cooling tower.
(*vi*) Chilled water and condenser water pump sets.
(*vii*) Air distribution ducting and grills.

Chilled water is produced in the refrigeration plant housed in the main plant room. Chilled water is then carried through insulated chilled water piping to air-handling unit or fan coil unit, where the return-air and the fresh-air mixture is filtered, cooled and dehumidified and then distributed to the conditioned space through galvanized iron and aluminium ducting and grills/diffusers. The return air is brought back to the air-handling unit through the annular space formed around the supply air duct and the false ceiling enclosure. Here it is mixed with the fresh air or ventilation air and then passed through the filters and the cooling coil.

Selection of air-conditioning system

The selection of correct air-conditioning system for a particular building depends upon the following factors :

(*i*) Capital cost.
(*ii*) Running and operating cost.
(*iii*) Space for the location of the equipment.
(*iv*) Type of application : Whether comfort application or industrial application.
(*v*) Type of controls required : Whether rigid control on the inside of the conditioned space is required as in the case of some industrial applications.
(*vi*) Acoustic considerations, such as in auditoriums, radio and TV studios, conference rooms, etc.
(*vii*) Type of filtration, whether sterile room or clean room application.

Table 27.2 gives general guidelines for the selection of proper air conditioning system for various applications. In the table, √ indicates the recommonded type of air conditioning system.

TABLE 27.2. SELECTION OF AIR CONDITIONING SYSTEM

	Application	*Room air conditioner*	*Packaged unit*	*Control DX plant*	*Chilled water plant*
1.	Small offices	√	√	—	—
2.	Small shops and restaurants	√	√	—	—
3.	Residential flats and small houses	√	√	—	—
4.	Small operation theatres	—	√	—	—
5.	General ward and private ward of a small hospital	√	√	—	—
6.	Cinema halls and auditoriums	—	—	√	√
7.	Medium type offices	—	√	√	—
8.	Large multi storeyed office buildings	—	—	—	√
9.	Small Laboratories	—	√	√	—
10.	Small hotels	√	√	—	—
11.	Large hotels : four star and five star	—	—	—	√
12.	Small Pharmaceutical factory	—	√	√	—
13.	Large Pharmaceutical factory	—	√	√	—
14.	Large size industrial application	—	—	—	√

27.9. ESSENTIALS OF AIR CONDITIONING SYSTEM

The following are the essentials of an air conditioning system;

1. Filtration.
2. Heating (in winter season).
3. Cooling (in summer season).
4. Humidification.
5. Dehumidification.
6. Air circulation or distribution.

1. Filtration

The aim of the filters is to exclude from incoming air dust particles, ash, chemical soot, bacteria and other micro-organisms, so that clean air is obtained. The filters should possess the following qualities :

(*i*) They should be capable of removing dust, ash, chemical soot, bacteria etc., from the incoming air.

(*ii*) They should be capable of holding a moderate amount of dust, cleaned from incoming air, on their surface without affecting their working efficiency. Filtering media should be of some fibrous material such as spun glass, steel wool, porous paper, wood fibre, etc., so that dust can adhere to it.

(*iii*) They should offer low frictional resistance to the flow of air.

(*iv*) They should be workable under a sufficient range of air velocities.

(*v*) They should afford easy cleaning, either manually or automatically.

Types : Filters may be of the following types :

(*i*) *Viscous type filters*. These are made of mats or screens of split wire or glass wool or of similar material, and coated with non-drying viscous oil so that dust in the incoming air can be caught and removed. They may be either of *unit type* or *automatic type*. In the unit type, the mats are replaced for cleaning and recoiling. In the automatic type an endless moving chain is provided over the mat so that it is mechanically rotated in a continuous cycle of air cleaning, removal of air-pollution particles and replacement of viscous filter media.

(*ii*) *Dry filter*. They are made of cloth such as flannel, cellulose, felt, etc., which is discarded when it becomes dirty.

(*iii*) *Spray washers*. The incoming air is allowed to pass through water sprays, where the dust and fumes are removed by drops of water.

(*iv*) *Electric precipitators*. These remove the dust by subjecting it to a strong electric field and then getting it attached to negative electrode. The particles collected on negative electrode are removed at intervals. The initial installation cost of this is high but operational cost is low.

2. Heating

Heating of air is necessary in winter, so as to compensate the heat loss from the room. Pre-heating of incoming air may be done by passing it over warm air furnaces, or by coils around which hot water or steam is circulating.

3. Cooling

Cooling of incoming air is necessary in summer. As stated previously, there are two methods of cooling the air (*i*) direct expansion system, and (*ii*) chilled water system. In the direct expansion (DX) system, the principle of *mechanical refrigeration* is used, where in a volatile refrigerant is compressed, cooled, allowed to expand and

then it is passed through coils. These coils absorb the heat from incoming air. In the chilled water system, a secondary medium such as water is utilised to cool the air. The incoming air is circulated around the coils containing chilled water. Thus, the refrigerant first cools the water and then the water cools the air. Chilled water is produced in the refrigeration plant housed in the main plant room.

4. Humidification

Humidification or addition of water to air is necessary in winter when air, because of its low temperature, has very low humidity. For ordinary conditions, humidification can be done by allowing the incoming air to pass through pans of water or wetted cloth strips. However, when large volumes of air is handled, humidification is accomplished by *spray humidifiers*. In the latter case it is essential to instal eliminator plates arranged in zig-zag manner so that fine water drops contained in humidified air are removed. Excess humidification should be avoided since it results in condensation on room surfaces.

5. Dehumidification

In this process certain reqiured amount of water is extracted from air. This is done in summer when the incoming air is cooled and dehumidified before the entry into the room. Its accomplished by (*i*) condensation, or by (*ii*) desiccation. In the former method, the temperature of air is first brought dowen below the dew point and then condensing out the required quantity of moisture from it ; the air is then reheated to the desired temperature with dry heat. In the desiccation method either absorbants are used or adsorbants are used. The *absorbants*, such as the solutions of salts of calcium or ammonia, possess the capacity to absorb excess moisture from air. The air is passed through the beds of small particles of these absorbants and is thus dehumidified. The *adsorbants* such as silica jels and activated alumina, allow moisture to stay on their surfaces. These absorbants can be reactivated by removing the collected moisture from their surfaces by heating.

6. Air circulation or distribution

This is one of the most essential requirement and the efficiency of the system depends largely on the air circulation or distribution system. The minimum fresh air requirements, as recommended by Indian Standard, are given in Table 27.3. Air circulation is achieved by the following : (*i*) air pumps, (*ii*) air delivery system consisting of supply and return ducts, and (*iii*) air distribution system consisting of inlets and outlets.

(*i*) *Air pumps*

Air pumps may be of two types : (*a*) Axial flow, propeller or fan type ; and (*b*) Radial flow, centrifugal or blower type. Propeller type pumps are used to produce air at relatively low pressures. They

have low initial cost and low operationa cost but produce excessive noise. Centrifugal fans are very suitable for air conditioning since they have low speed, quite operation and large capacity. The type of the fan should be so selected that it can give to the air the needed velocity and pressure to overcome to the overflow set up by the ducts and to maintain desired velocity at the exit from the supply grills.

(*ii*) *Air delivery system*

This consists of the following components : (*a*) supply ducts, (*b*) return ducts, (*c*) dampers, and (*d*) duct insulation. The supply ducts as well as return ducts are made of sheet metal of the required size. The supply duct, properly insulated against the heat loss, are carefully shaped and designed so that proper velocity and pressures are maintained. Dampers are installed in the duct, and are operated either manually or automatically, to control the direction, velocity and volume of circulating air.

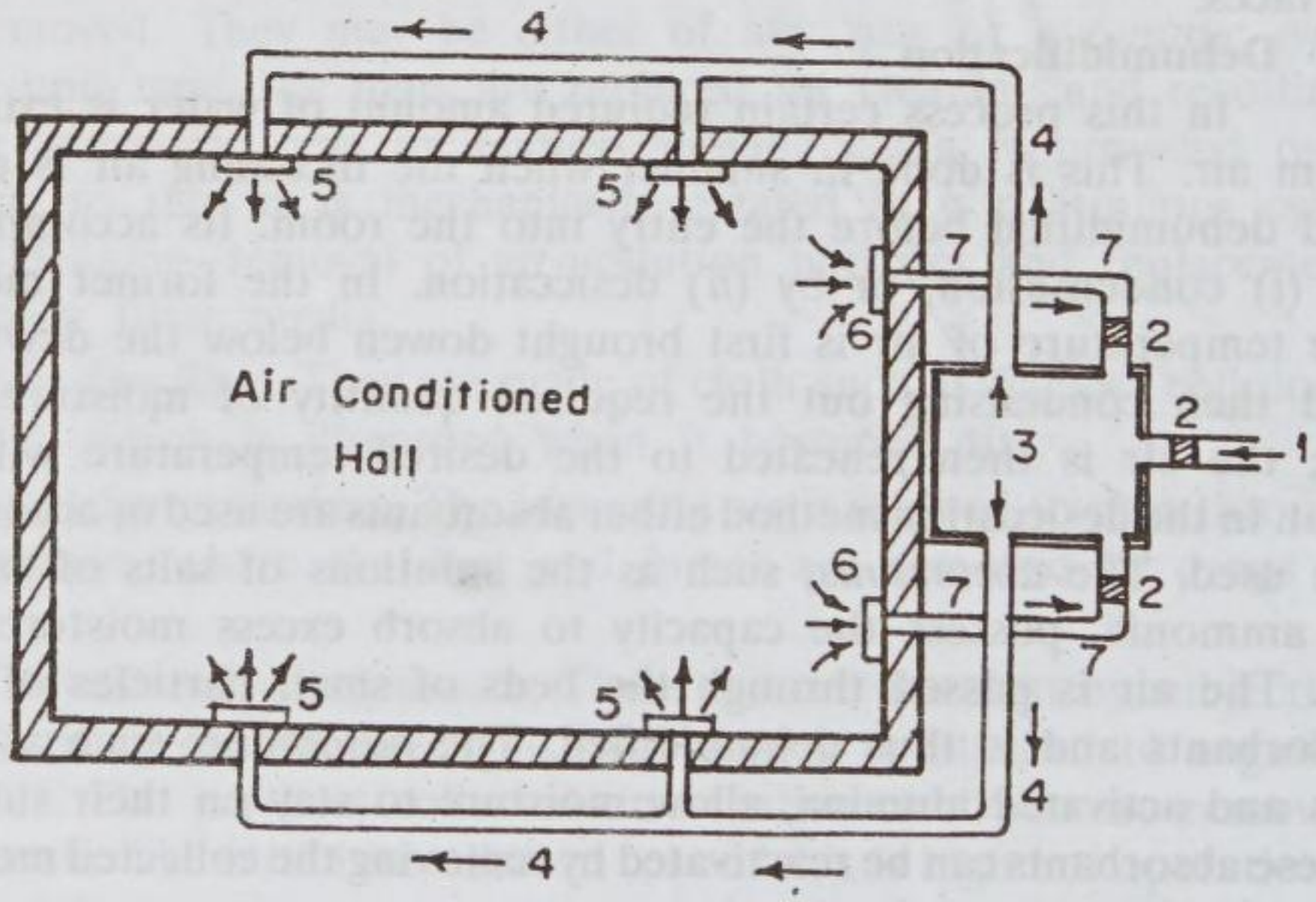


FIG. 27.6. LAYOUT OF AIR-CONDITIONED HALL, SHOWING AIR CIRCULATION

1. INCOMING VENTILATION AIR 2. DAMPERS. 3. AIR CONDITIONER
4. SUPPLY DUCT. 5. OUTLET GRILLS. 6. RETURN AIR INLET.
7. RETURN AIR DUCT. 8. EXHAUST.

(*iii*) *Air distribution system*

This consists of *outlets* for the supply of conditioned air into the room, and *inlets* for the collection of return air, each in the form of grills registers. The outlets are placed at a height of not less than 2 m above the floor level, nor less than about 45 cm from the ceiling. These should distribute the air without any draught. Inlets are provided for the collection of return foul air from the conditioned space.

TABLE 27.3. MINIMUM FRESH AIR REQUIREMENTS

Application	*Smoking*	*m^3/min. per person*		*m^3/min. per m^3 of floor area*
		Recommended	*Minimum*	
1. Apartments	Some	0.56	0.28	—
2. Banking space	Occasional	0.28	0.21	—
3. Board rooms	Very heavy	1.40	0.56	—
4.Departmental stores	None	0.21	0.14	0.015
5. Director's room	Very heavy	1.40	0.84	—
6. Drugs room	Considerable	0.28	0.21	—
7. Factories	None	0.28	0.21	0.30
8. Garrages	—	—	—	0.30
9. Hospitals				
(a) Operating rooms (All fresh air)		—	—	0.60
(b) Private rooms	None	0.84	0.70	0.10
(c) Wards	None	0.56	0.28	—
10. Hotel rooms	Heavy	0.84	0.70	0.10
11. Kitchens				
(a) Restaurant	—	—	—	1.20
(b) Residence	—	—	—	0.60
12. Laboratories	Some	0.56	0.42	—
13. Meeting rooms	Very heavy	1.40	0.84	0.38
14. Offices				
(a) General	Some	0.42	0.28	—
(b) private	None	0.70	0.42	0.08
15. Restaurants				
(a) Cafeteria	Considerable	0.34	0.28	—
(b) Dining room	Considerable	0.42	0.34	—
16. Retail shop	None	0.28	0.21	—
17. Theatre	None	0.21	0.14	—
	Some	0.42	0.28	
18. Toilets (exhaust)	—	—	—	0.60

PROBLEMS

1. What do you understand by (*a*) ventilation, (*b*) air conditioning? Explain the necessity of each.

2. Discuss in brief the functional requirements of ventilation system.

3. Describe in brief (*a*) wind effect, (*b*) stack effect. Write down the expression for rate of air flow in each case.

4. Write down general rules for natural ventilation.

5. Explain in brief various systems of mechanical ventilation.

6. Differentiate between comfort air conditioning and industrial air conditioning. Explain various controls required for comfort air conditioning.

7. Write notes on :

   (*i*) Filters

   (*ii*) Humidification and dehumidification, and

   (*iii*) Air circulation.

28

Acoustics and Sound Insulation

28.1. INTRODUCTION

'Acoustics' is the science of sound, which deals with origin, propogation and auditory sensation of sound, and also with design and construction of different building units to set optimum conditions for producing and listening speech, music, etc. The knowledge of this science is necessary for the proper functional design of theatres, cinema halls, auditoriums, conference halls, hospitals, etc. so that unwanted sound is excluded or insulated.

Sound is generated in the air when a surface is vibrated. The vibrating surface sets up waves of compression and rarefaction in the air and these set the ear drum vibrating. The movements of the ear drum are translated by the brain into sound sensation. When the sound waves are periodic, regular and long continued, they produce a pleasing effect ; such a sound is known as *musical sound*. On the contrary, when the sound wave is non-periodic, irregular and of very short duration, it produces displeasing effect ; such sound is known as *noise*. A noise is an abrupt sound of complex character with an irregular period and amplitude originating from a source of non-periodic motion.

28.2. CHARACTERISTICS OF AUDIBLE SOUND

Sound is transmitted in the form of waves which are a series of compressions and rarifications created in the medium through which it travels. The sound waves are longitudinal waves and hence each particle of the medium through which sound wave is proceeding, moves backwards and forwards along a line in the direction in which sound is travelling. The velocity of sound depends upon the nature and temperature of the *medium* through which it travels. It travels much faster in solids and liquids than in air. The velocity of sound

in air depends upon moisture in air and temperature of air. The velocity of sound in atmospheric air at 20°C is 343 m/sec. The velocity of sound in pure water is 1450 m/sec while that in bricks and concrete is 4300 and 4000 m/sec respectively. Sound cannot travel in vacuum. For the sound to be audible, the sound source and ear must be connected by an uninterrupted series of portions of elastic matter.

There are three characteristics of sound : (*a*) Intensity and loudness, (*b*) Frequency and pitch, and (*c*) Quality or timbre.

1. Intensity and loudness of sound

Intensity of sound is defined as the amount or flow of wave energy crossing per unit time through a unit area taken perpendicular to the direction of propagation. Mathematically, the energy of a wave and hence the intensity at a point is proportional to the square of the amplitude of vibration of the point, *i.e.* $I \propto A^2$. But the distinction between the physical quantity called *intensity* and the meaning to be understood by the term *loudness* must be clearly noted. Loudness of a sound corresponds to the degree of sensation depending on the intensity of sound and the sensitivity of ear drums, and does not increase proportionally with intensity but more nearly to its logarithm, i.e., $L \propto \log I$. It is know as Weber and Fechner's law which states that the magnitude of any sensation is proportional to the logarithm of the physical stimulus that produces it. Thus, *intensity* of sound is purely a physical quantity which can be accurately measured, and which is independent of ear of listener. *Loudness*, on the other hand, is the degree of sensation which is not wholly physical but partly subjective and does depend upon the ear and the listener. It may also happen that the same listener might give different judgements about the loudness of sounds of the same intensity but of different frequencies as the response of the ear is found to vary with the frequency of vibration.

The range of variation of intensity is very large. The loudness of a sound as judged by the ear is proportional to the logarithm of intensity. If I_1 and I_0 represent the intensities of two sounds of a particular frequency, and L_1 and L_0 are their corresponding measures of loudness, we have

$$L_1 = k \log_{10} I_1$$

and

$$L_0 = K \log_{10} I_0$$

The difference in loudness of the two, technically known as *intensity level* L between them, is given by

$$L = k \log_{10} \frac{I_1}{I_0} \qquad ...(28.1)$$

In the above equations, k is the constant depending upon the units of measurement. When $k = 1$ (unity,) the difference in loudness is expressed in bels, a unit named after A.G. Bel. This unit is rather

large. Hence a shorter practical unit called *decibel* (written as dB) equal to 1/10 of be, is used. Thus, the intensity level is expressed as

$$L = 10 \log_{10} \frac{I_1}{I_0} dB \qquad ...(28.2)$$

If $L = 1\, dB$,. we have

$$1 = 10 \log_{10} \frac{I_1}{I_0}$$

or $$\log_{10} \frac{I_1}{I_0} = \frac{1}{10}$$

$$\therefore \quad \frac{I_1}{I_0} = 1.26$$

i.e., a 26 percent change in intensity alters the level by one decibel. This is practically the smallest change in intensity level that the ear can ordinarily detect.

Also, when $I_1 = 100\, I_0$, we get

$$L = 10 \log_{10} 100 = 10 \log_{10} 10^2 = 20\, dB.$$

Similarly, when $I_1 = 1000\, I_0$, we have

$$L = 10 \log_{10} 1000 = 10 \log_{10} 10^3 = 30\, dB.$$

Thus, we learn that when two sounds differ by 20 dB, the louder of them is 100 times more intense and when they differ by 30 dB, the louder one is 1000 times more intense.

To build a scale of loudness, we have to fix its zero. The loudness corresponding to the *threshold of hearing* is the zero of this scale, while 130 dB is the threshold of painful hearing. Table 28.1 gives the rating of intensity of sound, in decibels :

TABLE 28.1. RATING OF INTENSITY OF SOUND

Common sound	*Intensity level (dB)*		*Threshold of feeling*
	Range	*Average*	
1. Threshold of audibility.		0	Very very faint
2. Rustle of leaves, whisper, sound proof room.	0—20	10	Very faint
3. Quiet living room, private office, quiet conversation, average auditorium.	20—40	30	Faint
4. Noisy home, average office (acoustically treated), average conversation , quiet radio etc.	40—60	50	Moderate
5. Noisy office, average street noise, average radio, average factory	60—80	70	Loud
6. Noisy factory area, loud street noise, police whistle, truck unmuffled, train sound.	80—100	90	Very loud
7. Thunder, artillary, air plane motors, pneumatic hammers etc.	100—120	110	Deafening
8. Loudest sound due to pneumatic drills, or aeroplane at a distance of 4 m	120—140	130	Pain and discomfort

The sound pressure corresponding to the threshold of hearing is about 0.0003 dynes/sq. cm and that corresponding to threshold of pain is about 300 dynes/sq. cm.

Table 28.2 gives acceptable indoor noise levels for various buildings.

TABLE 28.2. ACCEPTABLE INDOOR NOISE LEVELS

Type of Building	*Noise level range dB*
1. Radio and T.V. studios	25—30
2. Music room	30—35
3. Hospital and auditoria	35—40
4. Apartments, hotels and homes	35—40
5. Conference rooms, small offices and libraries	35—40
6. Court rooms and class rooms	40—45
7. Large public offices, banks and stores	45—50
8. Restaurants	50—55

2. Frequency and pitch of sound

Frequency or Pitch is defined as the number of cycles which a sounding body makes in each unit of time. It is a measure of the *quality* of a sound. It is that characteristic by which a shrill sound can be distinguished from a grave one, even though the two sounds may be of the same intensity. The *sensation of pitch* depends upon the *frequency* with which the vibrations succeed one another at the ear ; *the greater the frequency the higher the pitch and the lesser the frequency the lower the pitch*. The frequency scale covers a wide range varying from 20 cycles per second to 1500 cycles per second.

3. Quality or Timbre

The quality of a sound is that characteristic which enables us to distinguish between two notes of the same pitch and loudners played on two different instruments or produced by two different voices. A study of vibration curves of various musical instruments has shown that the notes emitted by them are seldom pure. They contain some fundamental *tones* of frequency n and additional tones of (of frequencies $2n$, n, $4n$, etc.) called *overtones*. The quality of a note is determined by its complex structure and depends upon the presence or absence of a certain number of overtones, on their relative strengths and pitches. It is to be noted that it is the memory of this *tonal quality* which enables us to recognise a large number

of different sounds. Among these are the voices of friends and acquaintances, the various sounds employed in speech and familiar musical instruments and the cries of animals.

28.3. BEHAVIOUR OF SOUND IN ENCLOSURES

When sound is generated in a room, the distance between the source and the walls is so small that there is little or no reduction due to distance. When the sound waves strike the surfaces of a room, three things happen :

(*i*) Some of the sound is *reflected* back in the room.
(*ii*) Some of the sound energy is *absorbed* by the surfaces and listeners.
(*iii*) Some of the sound waves set on the walls, floors and ceiling vibrating and are thus *transmitted* outside the room.

The amount of sound *reflected* or *absorbed* depends upon the surfaces, while the sound *transmitted* outside the room depends upon *sound insulation* properties of the surfaces.

28.4. REFLECTION OF SOUND

Sound waves get reflected from a large uniform plane surface in the same manner as that of light waves, the angle of incidence being equal to angle of reflection, as shown in Fig. 28.1. The reflection of sound has certain virtues in acoustics, such as the enhancement

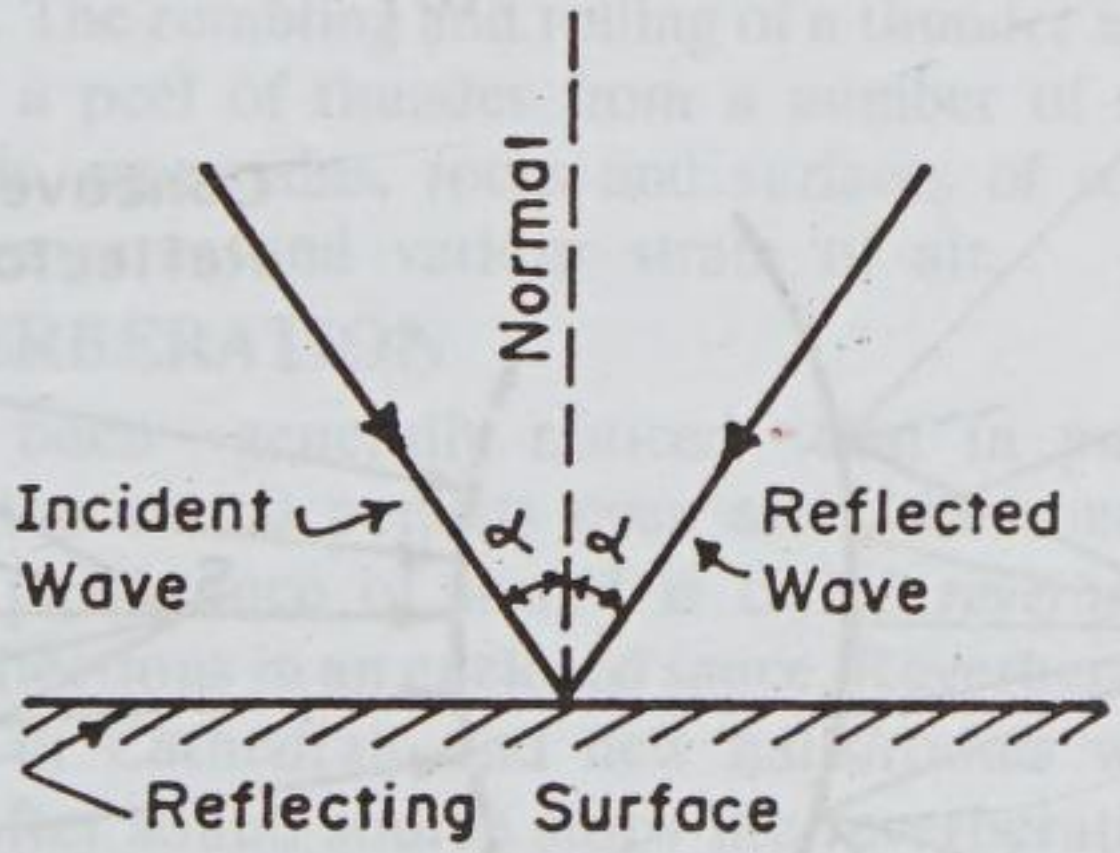


FIG. 28.1. REFLECTION OF SOUND WAVES.

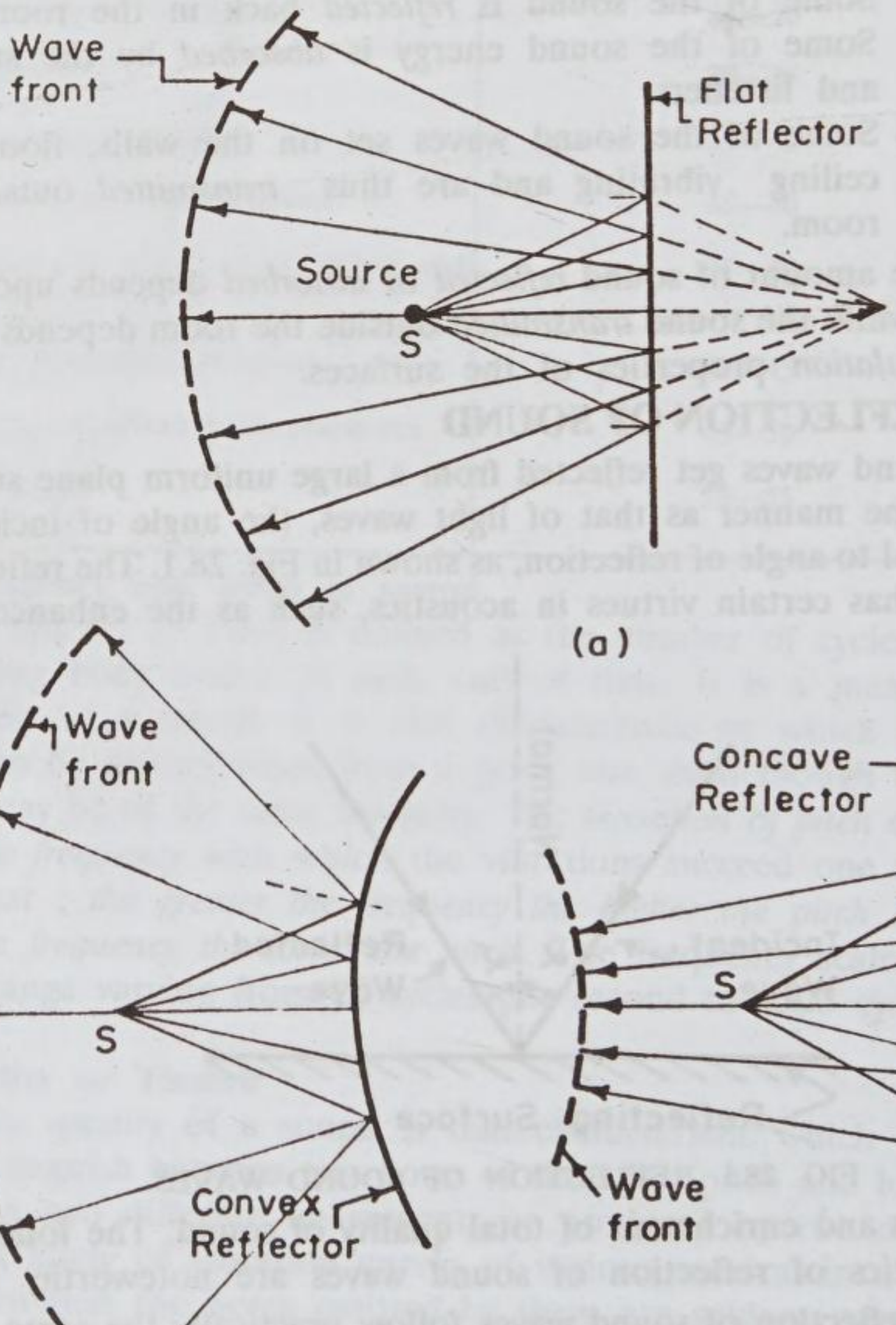
of loudness and enrichment of total quality of sound. The following characteristics of reflection of sound waves are noteworthy :

1. Reflection of sound waves follow practically the same laws as reflection of light. However, this may not be true in some exceptional cases, hence great caution should be exercised while applying these laws.

2. The reflected wave fronts from a flat surface are also spherical and their centre of curvature is the image of source of sound Fig. 28.2 (*a*).

3. Sound waves reflected at a convex surface are magnified and are considerably bigger Fig. 28.2 (*b*). They are attenuated and are therefore weaker. Convex surfaces may be used with advantage to spread the sound waves throughout the room.

4. The sound waves reflected at a concave surface are considerably smaller Fig. 28.2 (*c*). The waves are most condensed and therefore amplified. The concave surfaces may be provided for the concentration of reflected waves at certain points.



(*a*) FROM FLAT SURFACE (*b*) FROM CONVEX SURFACE (*c*) FROM CONCAVE SURFACE

FIG. 28.2. REFLECTION OF SOUND WAVES.

Defects due to reflected sound

The behaviour of reflected sound plays very important role in the acoustical design of an enclosed space. The following are two main defects that may be caused due to reflection of sound waves :

(*a*) Echoes

(*b*) Reverberation.

Echoes

An *echo* is produced when the reflected sound wave reaches the ear just when the original sound from the same source has been already heard. Thus, there is repetition of the sound. The sensation of sound persists for $\frac{1}{10}$th of a second after the source has ceased. Hence in order that an echo may be distinguished as separate, it must reach the ear $\frac{1}{10}$th of a second after the direct sound. Taking the velocity of sound as 340 m/sec, it means that sound must come after traversing a distance of 34 m, *i.e.*, the minimum distance of the obstacle from the source must be half of this, *i.e.*, 17 m. If, however, the distance of the reflecting surface is less than this, the sound will appear to be *draw out*. Near echoes, sufficient to cause blurring, occur when the distance of the reflecting surface is between 8 and 17 m. *Multiple echoes* may be heard when a sound is reflected from a number of reflecting surfaces suitably placed, such as two parallel cliffs. The rumbling and rolling of a thunder is due to successive reflections of a peel of thunder from a number of reflecting surfaces such as clouds, mountains, rocks and surfaces of separation between atmospheric currents and various strata of air.

28.5. REVERBERATION

It has been generally noticed that in public halls and auditoriums, the sound persists even after the source of sound has ceased. This persistence of sound is called *reverberation*. It is due to multiple reflections in an enclosed space. Reverberation is a faimiliar phenomenan in Cathedrals and new halls/rooms without furniture, where, even after sound source stops the reverberation is heard even upto 10 seconds. A certain amount of reverberation is desirable, specially for giving richness to music, but too much reverberation is undesirable.

The time during which the sound persists is called the *reverberation time* of sound in the hall. It is the time taken by the reverberant sound to decay to its one-millionth of the sound intensity level existing at the time the source of sound stopped. In other words, it is the period of time in seconds, which is required for sound energy to decay or diminish by 60 dB after the sound source has stopped.

Sabine's expression for reverberation time

Prof. W.C. Sabine (1868-1919) of Hardward University studied the whole subject of architectural acoustics, particularly with reference to reverberation time. He found experimentally that the reverberation time of a room varies inversely as the effective surface area and directly as the volume of the room. He also showed that this time is independent of the position of the source and the listener and the shape of the room.

As the result of the experiments, he established the following expression for reverberation time :

$$t = \frac{0.16\,V}{\alpha_1 s_1 + \alpha_2 s_2 + \alpha_3 s_3 ....} \quad ...(28.3)$$

or $$t = \frac{0.16\,V}{\Sigma\, ds} = \frac{0.16\,V}{A} \quad ...(28.4)$$

where t = Reverberation time in seconds

V = volume of the room in m^3

$\alpha_1, \alpha_2, \alpha_3 ...$ = Absorption coefficient of individual units (*i.e.* walls, floors, ceilings, etc.) See Table 28.5.

$s_1, s_2, s_3 ...$ = Area of individual absorbings surfaces

A = Total absorbing power.

The total *absorbing power* is expressed in m^2 sabines.

Fig. 28.4 is also used to calculate the total absorption to be provided, in order to achieve any desired time of reverberation.

Table 28.3 gives the relation between reverberation time and the acoustics of a room.

TABLE 28.3. REVERBERATION TIME AND ACOUSTICAL QUALITY

Reverberation time in seconds	*Acoustics*
0.50 to 1.50	Excellent
1.50 to 2.00	Good
2.00 to 3.00	Fairly good
3.00 to 5.00	Bad
Above 5.00	Very bad

Table 28.4 gives the optimum reverberation time and audience factors for acoustical design.

Indian Standard Code IS : 2526-1963 recommends to use Fig. 28.3 for the determination of reverberation time for various size of enclosed space and for various purpose/use of the space.

TABLE 28.4. OPTIMUM REVERBERATION TIME

Type of building	*Optimum reverberation time (seconds)*	*Audience factor*
1. Cinema theatres	1.3	Two-Thirds
2. Churches	1.8 to 3	Two-thirds
3. Law courts, committee rooms, conference halls	1 to 1.5	One-third
4. Large halls	2. to 3	Full
5. Music concert hall	1.6 to 2	Full
6. Parliament house, Assembly hall, Council chamber	1 to 1.5	Quorem
7. Public lecture hall	1.5 to 2	One-third

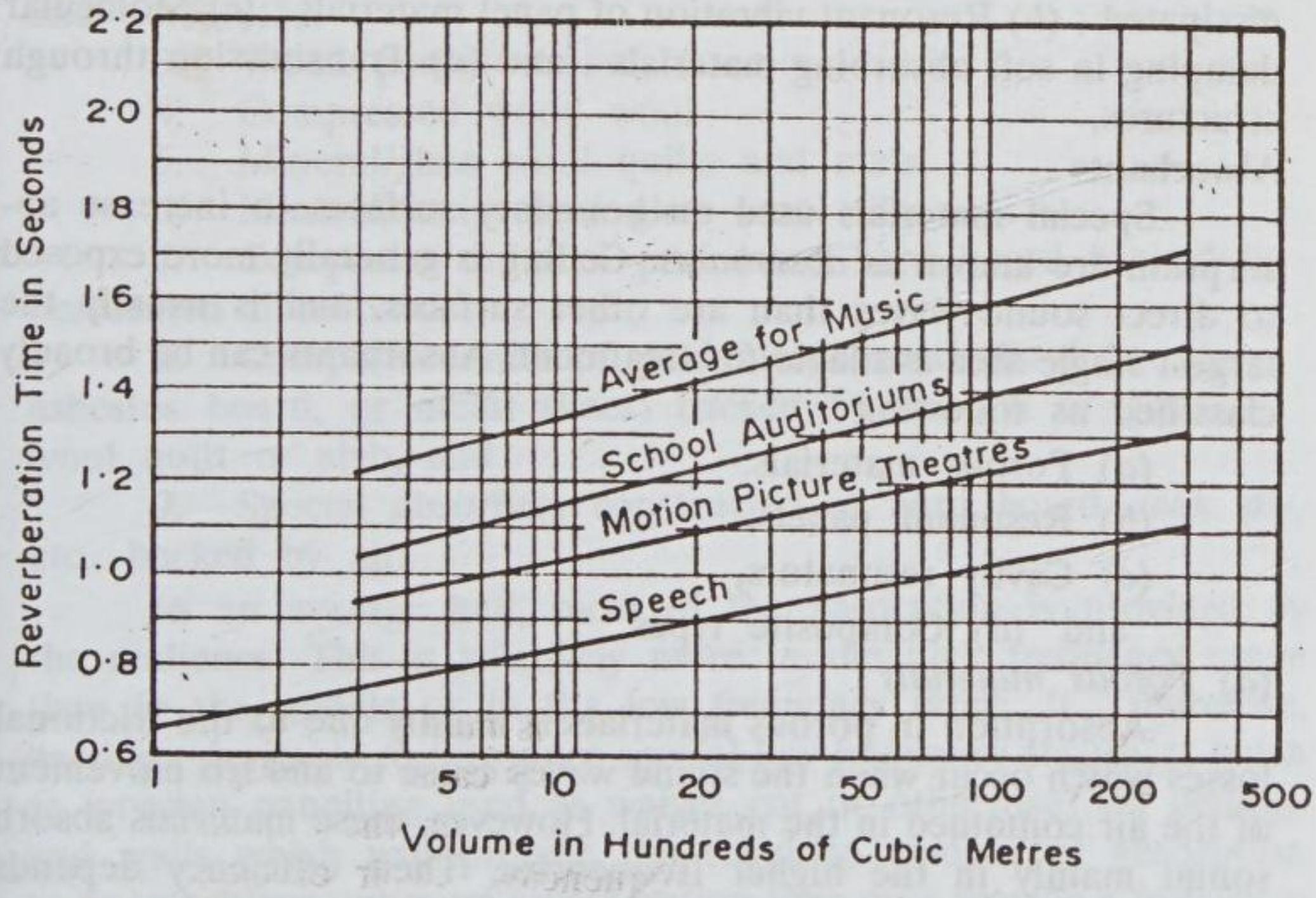


FIG. 28.3. OPTIMUM REVERBERATION TIME AT 500 CYCLES FOR DIFFERENT TYPES OF ROOMS AS A FUNCTION OF ROOM VOLUME.

28.6. ABSORPTION

When a sound wave strikes a surface, a part of its energy is absorbed by friction. The sound generated in an auditorium or hall is absorbed in four ways : (*i*) in the air, (*ii*) by the audience, (*iii*) in furniture and furnishing, and (*iv*) at the boundary surfaces such as floors, ceilings, walls, etc.

Absorption in the air

The absorption of sound in the air is mainly due to the friction

between the oscillating molecules when sound wave travels through it. However, this absorption is extremely small.

Absorption by the audience

Sound energy absorbed by the clothing of the audience. Room acoustics change perceptibly by the number of audience present. Also, absorption is more in winter, than in summer, because of heavy clothings.

Absorption in furniture and furnishings

Furniture, curtains, carpets, etc. also absorb sound energy to a fairly good extent.

Absorption by boundary surface

When sound waves strike the boundary surfaces such as walls, floors, ceilings (treated or otherwise), absorption takes place due to the following factors : (*a*) Penetration of sound into porous materials, causing resonance within air pockets in the pores until energy is dissipated ; (*b*) Resonant vibration of panel materials ; (*c*) Molecular damping in soft absorbing materials ; and (*d*) Transmission through structures.

Absorbants

Special materials used on boundary surfaces to increase absorption are known as *absorbants*. Ceiling is generally more exposed to direct sound waves than are other surfaces, and is usually the largest single area available for treatment. Absorbants can be broadly classified as following :

(*a*) Porous materials,
(*b*) Resonant panels,
(*c*) Cavity resonators,
and (*d*) Composite types.

(*a*) *Porous materials*

Absorption in porous materials is mainly due to the frictional losses which occur when the sound waves cause to and fro movement of the air contained in the material. However, these materials absorb sound mainly in the higher frequencies. Their efficiency depends upon porosity, the resistance to air flow through the materials and the thickness. Examples of absorbants under this category are rock wool, glass silk, wood wool, curtains .nd other soft furnishings ; drilled fibre boards and acoustic plasters.

(*b*) *Resonant panels*

These panels absorb the sound by damping the sympathetic vribrations in the panels, caused by sound pressure waves of appropriate frequency, by means of air space behind the panel. These panels absorb sound only at lower frequencies, over a comparatively narrow frequency band ranging from 50 to 200 cycles. The frequencies at which panels vibrate depend upon their weight and depth of air

spaces behind them.

(*c*) *Cavity resonators*

A cavity resonator is virtually a container with a small opening, and it functions by the resonance of air in it. They can be designed to absorb sound of any frequency.

(*d*) *Composite absorbers*

These are a comparatively recent development, combining the functions of all the above three absorbents. It consists of a perforated panel fixed over an air space containing porous absorbent. The perforations in the panel should form atleast 10 percent of the total area to allow the porous materials to absorb sound at higher frequencies.

Following are the sound absorbing materials commonly used:

1. Acoustic plaster (a plaster which includes granulated insulation material with cement).
2. Compressed cane or wood fibre board, unperforated and perforated.
3. Wood particle board.
4. Compressed wood wool
5. Mineral/glass wool quilts and mats.
6. Mineral/compressed glass wool tiles.
7. Composite units of perforated hard board backed by perforated fibreboard.
8. Composite units of perforated board (hard board, asbestos board, or metal sheet) backed by mineral or glass wool quilt or slab, and
9. Special absorbers constructed of hard board, teak ply, etc., backed by air.

In an average hall, most of the absorption is provided by the audience. This is relatively more in the high frequency range than in the middle or in the low frequency range. It , therefore, becomes desirable to introduce special low frequency absorbers (such as wooden panelling used as waives cot or otherwise) on ceilings and walls which will provide the requisite amount of absorption so as to achieve optimum reverberation time over as wide a frequency range as possible.

Those areas which cause objectionable sound reflection and need to be treated with sound absorbents should be earmarked for treatment with sound absorbing material. These areas are : (*i*) rear wall, (*ii*) balcony parapet, (*iii*) any area which may reflect sound back to the stage, (*iv*) concave areas which have a tendency to focus sound in certain places, and (*v*) such other areas as will contribute to indirect sound arriving at any point in the hall later than 50 milli-seconds after the direct sound. The rest of the sound absorbing material required to be introduced in the room should be distributed

over the various remaining surfaces.

Absorption coefficients and measurement of absorption

The sound reducing effect of an absorber depends upon its area as well as on the efficiency of the material, and is indicated by a sound *absorption coefficient*. This absorption ratio of a surface is the ratio of sound energy absorbed to the incident sound energy on a material.

The unit of absorption is the *open window unit* which is called a *sabin*, named after the scientist who established the unit. A m^2-*sabine* is the unit of sound absorbed by one square metre area of fully open window. The ratio of the sound absorbed by one square metre of any surface to that absorbed by one square metre of open window is called *coefficient of absorption* for that surface. In other words, the absorption coefficient of an open window is taken as unity, assuming that sound wave will completely pass through it. *The absorption of a surface is the product of the area of the surface multiplied by its absorption coefficient* and is expressed in m^2 sabins.

The total absorption A of a hall or auditorium will be the sum of (*a*) product of the volume of air and its coefficient of absorption per cubic metre ; (*b*) product of surface area of each absorbent surface and their corresponding coefficients of absorption ; (*c*) product of unoccupied seats by the coefficient of each unoccupied upholstered seat, and (*d*) product of number of persons present in the hall and the average coefficient of absorption person. This value of total absorption A, expressed in m^2-sabins, is then substituted in Eq. 28.3 or 28.4 to compute the reverberation time of the hall. If this computed time is not within desirable limits (Table 28.4), the total absorption A is changed to bring the time t within the desirable limits.

Extensive work has been done to determine co-efficient of absorption of various materials. This coefficient depends upon the frequency of sound. An average absorption coefficient is usually an average of low, medium and high frequencies, *i.e.*, 125, 500 and 2000 cycles per second.

Table 28.5 gives absorption coefficients for building materials and furnishing. Table 28.6 gives absorption coefficient of indigenous acoustical materials.

Requirements of a good acoustic material

1. It should have high coefficient of absorption.
2. It should be efficient over a wide range of frequencies.
3. It should be relatively cheap and easily available.
4. It should give pleasing appearance after fixing.
5. It should be self supporting, and should afford easy fixing.

TABLE 28.5. ABSORPTION COEFFICIENTS FOR BUILDING MATERIAL AND FURNISHINGS

S.No.	*Materials*	*Absorption coefficient at*		
		125 c/s	*500 c/s*	*2000 c/s*
	(a) Hangings and Floorings			
1.	Carpet, lined	0.10	0.25	0.40
2.	Carpets, unlined	0.08	0.15	0.25
3.	Cotton fabric, 475 g/m^2 draped to half its area	0.07	0.49	0.66
4.	Draperies, velours 610 g/m^2	0.05	0.35	0.38
5.	Draperies, as above draped to half their area	0.14	0.55	0.70
6.	Stage curtain	0.19	0.20	0.23
7.	Linoleum or concrete floor	0.02	0.03	0.04
8.	Floor, wood on solid	0.12	0.09	0.09
9.	Floor, wood boards on timber frame	0.25	0.13	0.15
	(b) Masonry and Building Materials			
10.	Brick wall 40 cm thick	0.02	0.03	0.05
11.	Plaster on wall	0.03	0.02	0.04
12.	Ceiling, 50 mm plaster of Paris suspended from trusses	0.08	0.05	0.04
13.	Plyboard on 75 mm air space	0.30	0.10	0.05
14.	Wood veneer 10 mm thick on 50×75 mm wood studs at 40 cm centre to centre	0.11	0.12	0.10
15.	Glass against solid surface	0.03	0.03	0.02
16.	Marble	0.01	0.01	0.01
	(c) Audience, Chairs etc.			
17.	Audience seated in fully uphoestered seats (per person)	0.18	0.46	0.51
18.	Chair, upholstered seat with spring	—	0.16	0.071
19.	Seat (unoccupied) fully upholstered (per seat)	0.16	0.40	0.44
20.	Wood veneer seat and back	—	0.023	—

TABLE 28.6. BSORPTION COEFFICIENTS OF INDIGENOUS ACOUSTICAL MATERIALS

S.No.	*Materials*	*Thick-ness (mm)*	*Density (g/cm^2)*	*Absorption coefficient at*		
				125 c/s	*500 c/s*	*2000 c/s*
1.	Fibrous (acoustic) plaster	20	0.1	—	0.30	0.50
2.	Compressed fibreboard:					
	(a) Unperforated	12	—	0.24	0.3	0.2
	(*b*) Perforated uniformly over part depth (rigid backing)	12.7	0.3	0.06	0.55	0.67
	(*c*) Peforated randomly over part depth (rigid backing)	12.7	0.3	0.15	0.52	0.76
3.	Compressed wood particle board					
	(*a*) Perforated (rigid backing)	12.7	0.37	0.04	0.36	0.78
	(*b*) Perforated (rigid backing)	19.1	0.34	0.05	0.61	0.91
	(*c*) Perforated and painted (rigid backing)	12.7	0.40	0.05	0.40	0.82
	(*d*) perforated and painted (rigid backing)	19.1	0.38	0.10	0.62	0.74
4.	(*a*) Wood wool board	25	0.4	—	0.20	0.60
	(*b*) Wood wool board (50 mm from wall)	25	0.4	—	0.35	0.35
5.	Mineral glass wool quilts and mats	25	0.06	0.09	0.17	0.50
6.	Bonded and compressed mineral/glass wool tiles	50	0.04	0.12	0.26	0.44
7.	Composite units of perforated hard board backed by perforated fibre board	25	0.4	0.25	0.5	0.65
8.	(*a*) Mineral/glass wool with scrim mat (rigid backing)	25	0.08	0.29	0.85	0.84
	(*b*) Minieral/glass wool with scrim mat (rigid backing)	50	0.08	0.57	0.99	0.95
	(*c*) Mineral/glass wool with scrim mat faced with perforated (10% open area) hard board (rigid backing)	25	0.08	0.06	0.99	0.49

TABLE 28.6. *(Contd.)*

S.No.	*Materials*	*Thick ness (mm)*	*Density (gm/cm^2)*	*Absorption coefficient at*		
				125c/s	*500c/s*	*2000 c/s*
9.	Miscellaneous :					
	(*a*) Straw board	13	0.24	—	0.30-	0.35
	(*b*) Straw board spaced 50 mm from wall	13	0.24	—	0.35	0.30
	(*c*) Composite panel : 5 mm perforated plywood, 50 mm mineral wool and 22 mm cement asbestos (suspended from the trusses)	—	—	0.36	0.95	0.67
	(*d*) Composite panel : 5 mm perforated plywood, 50 mm mineral wool and 22 mm hard board (suspended from trusses)	—	—	0.47	0.20	0.09

6. It should be fire resistant.
7. It should have sufficient structural strength.
8. It should be heat insulating and non-hygroscopic.
9. It should be durable, and should not be liable to attack by insects, vermits, termites etc.

28.7. COMMON ACOUSTICAL DEFECTS

Perfect acoustical conditions in a big room, hall or auditorium etc. are achieved when there is clarity of sound in every part of the occupied space. For this, the sound should rise to suitable intensity everywhere with no echoes or near echoes or distortion of the original sound ; with correct reverberation time. Following are the common defects which are encountered and which require special attention of the designer for proper treatment :

(1) Reverberation
(2) Formation of echoes
(3) Sound foci
(4) Dead spots
(5) Insufficient loudness
(6) Exterior noise.

1. Reverberation. We have already seen that reverberation is the persistance of sound in the enclosed space, after the source of sound has stopped. Reverberant sound is the reflected sound, as a result of improper absorption. Excessive reverberation is one of the most common defect, with the result that sound once created prolongs for a longer duration resulting in confusion with the sound created next. However, some reverberation is essential for improving

quality of sound. Thus, optimum clarity depends upon correct reverberation time which can be controlled by suitably installing the absorbent materials.

2. Formation of echoes. Echoes are also formed due to reflection of sound when the reflecting surfaces are situated at a distance greater than about 17 m and when the shape of the hall /auditorium/room is curved with smooth character. This defect can be removed by selecting proper shape of the hall and by providing rough and porous interior surfaces to disperse energy of echoes.

3. Sound foci. As indicated in Fig. 28.2 (*c*), reflecting concave surfaces cause concentration of reflected sound waves at certain spots, creating a sound of large intensity. These spots are called *sound foci*. This defect can be removed by (*a*) geometrical designed shapes of the interior faces, including ceilings, and (*b*) providing highly absorbent materials on focussing areas.

4. Dead spots. This defect is an outcome of the formation of sound foci. Because of high concentration of reflected sound at sound foci, there is deficiency of reflected sound at some other points. These points are known as *dead spots* where sound intensity is so low that it is insufficient for hearing. This defect can be removed by installation of suitable diffuser so that there is even distribution of sound in the hall.

5. Insufficient loudness. This defect is caused due to lack of sound reflecting flat surface near the sound source and excessive sound absorption treatment in the hall. The defect can be removed by providing hard reflecting surface near the source, and by adjusting the absorption of the hall so as to get optimum time of reverberation. When the length of the hall is more, it may be desirable to install loud speakers at proper places.

6. External noise. External noise from vehicles, traffic engines, factories, cooling plants etc. may enter the hall either through the openings (such as doors, windows, ventilaters etc.) or through even walls and other structural elements having improper sound insulation. This defect can be removed by proper planning of the hall with respect of its surroundings and by proper sound insulation of exterior walls.

Table 28.7 gives summary of various acoustical defects in auditoriums and conference halls and recommended remedies for the same.

28.8. ACOUSTICAL DESIGN OF HALLS

Conditions for good acoustics of a hall

The following conditions should prevail for halls possessing good acoustical properties :

1. The initial sound from the source should be of adequate

TABLE 28.7 SUMMARY OF COMMON ACOUSTICAL DEFECTS IN AUDITORIUMS AND CONFERENCE HALLS AND RECOMMENDED REMEDIES

S. No.	*Defect*	*Cause*	*Recommendations for*	
			New design	*Existing building*
1	Excessive Reverberation	Insufficient absorption	Add absorbents.	
2	Echoes	(*a*) Unsuitable shape (*b*) Remote reflecting surfaces	Avoid unsuitable shape. Make offending surfaces highly absorbent.	
3	Sound foci	Concave reflecting interior surfaces	Avoid curvilinear interiors.	Alter shape or use absorbents on focussing areas.
4	Dead spots	Irregular distribution of sound	Provide even diffusion of sound.	Introduce suitable diffusers
5	Insufficient sound volume	(*a*) Lack of reflections close to sourse to sound (*b*) Excessive absorption	Disperse hard reflecting surfaces about the source of sound. Adjust absorption to give optimum reverberation.	
6	Colouring of sound quality	(*a*) Selective absorption (*b*) Uncontrolled resonance	Use combination of absorbents to obtain uniform absorption coefficient over the required frequency range. Use wood panel absorbents which resonate over a wide frequency range and fix these on battens provided at irregular intervals. Adopt rigid construction with studs etc.	
7	High background noise	Poor sound insulation, badly fitting doors and windows or noisy air–conditioning system	Select construction with requisite sound insulation ; provide proper fitting doors and windows with requisite sound insulation. Reduce noise from air–conditioning equipment by isolating the machine and/or treatment of plant room etc.	

intensity so that it can be heard throughout the hall. For halls of big size, suitable sound amplification system should be installed.

2. The sound produced should be evenly spread in the hall so that sound foci and dead spots are avoided.

3. The boundary surfaces should be so designed that there are no echoes or near echoes.

4. The boundary surfaces of the hall should be properly designed so that the desired reverberation time is achieved, and unwanted sound is absorbed. The absorbent materials should be distributed evenly over the wall surfaces of the hall.

5. In the case of conference halls, the acoustics of the halls should be so designed as to ensure proper conditions for listening, assuming that a person may speak or listen from anywhere in the hall.

6. In the case of music halls, the treatment should be such that the initial sound reaches the audience with the same intensity and frequency.

7. The outside noise should be properly insulated.

General principles and factors in acoustical design

Following is the list of general planning principles and factors which are important for good acoustical conditions in a hall :

1. Site selection and planning.
2. Dimensions.
3. Shape.
4. Seats and seating arrangement.
5. Treatment of interior surfaces.
6. Reverberation and sound absorption.

1. Site selection and planning

There are many factors which are important for the site selection for an auditorium or hall, but problem of noise is an important consideration. A noise survey of the area should be made, and the site selected should be in quietest surroundings as otherwise elaborate and expensive construction may be required to provide requisite sound insulation. It is particularly necessary to keep the level of extranuous noise low by proper orientation site selection in cases where no air-conditioning is provided and doors and windows are normally kept open during performance. Where air-conditioning is provided, special care should be taken to attenuate the plant noise and the grill noise. Depending on the ambient noise level of the site, orientation, layout and structural design should be arranged to provide necessary noise reduction, so that the back ground noise level of not more that 40 to 45 dB is achieved within the hall.

2. Dimensions (size)

The size should be fixed in relation to the number of audience required to be seated, and also in proportion to the intensity of sounds to be generated. For music halls, the volume should be large so that enough space is available for the music to spread in the hall. On the other hand, for lecture halls, small volumes are useful for weak sounds.

The floor area of the hall, including gangways (excluding stage) should be calculated on the basis of 0.6 to 0.9 m^2 per person. The height of the hall is determined by such consideration as ventilation, presence (or absence) of balcony and type of performance. The average height may vary from 6 m for small halls to 7.5 m for large halls. Ceiling may be flat but it is preferable to provide slight increase in the height near the centre of the hall.

Suitable volumes for different types of auditoriums are recommended below :

(*a*) Public lecture halls	3.5 to 4.5 m^3/person
(*b*) Cinemas or theatres	4.0 to 5.0 m^3/person.
(*c*) Musical halls or concert halls	4.0 to 5.5 m^3/person.

3. Shape

The shape of a hall/auditorium is extremely important in the acoustical design since it is a governing factor in correcting defects like echoes, sound foci, dead spots, sound shadows etc. The shape of the hall is to be geometrically arranged in view of better audibility.

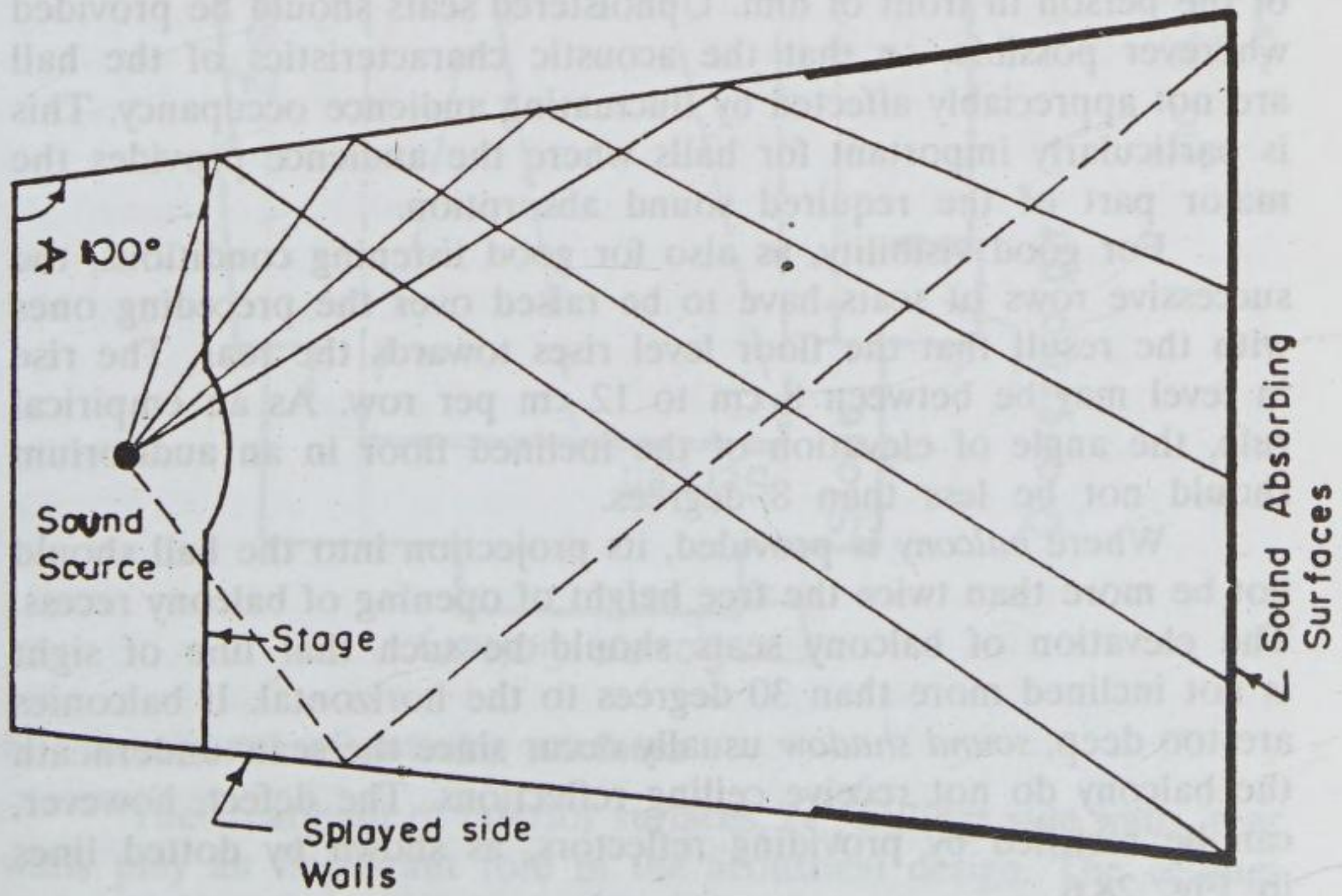


FIG. 28.4. FAN SHAPED PLAN FOR FAVOURABLE REFLECTION FROM SIDES

A fan shaped floor plan is preferred. The side walls should be arranged to have an angle of not more than 100 degrees with the curtain line. In the case of talking pictures, synchronisation of sound with lip movement is most essential. Also, in the case of theatres a person with normal vision should be able to discern facial expressions of the performers. In order to satisfy these conditions it is recommneded that the distance of the farthest seat from the curtain line should not normally exceed 23 metres.

4. Seats and seating arrangement

The seats should be arranged in concentric arcs of circles drawn with the centre located as much behind the centre of the curtain line as its (curtain line) distance from the auditorium rear wall.

The angle subtended with horizontal at the front-most observer by the highest object should not exceed 30°. On this basis the distance of the first row works to about 3.6 m for drama and it should be 4.5 m or more for cinema purposes. Minimum distance of front seats should be determined by the highest point required to be seen on the stage which is usually raised by 75 cm or more. The width of the seats should be between 45 cm and 56 cm. The back to back distance of chairs in successive rows of seats should be atleast 45 cm, and this may be increased upto 106 cm for extra comfort. Seats should be staggered sideways in relation to those in front so that a listener in any row is not looking directly over the head of the person in front of him. Upholstered seats should be provided wherever possible, so that the acoustic characteristics of the hall are not appreciably affected by fluctuating audience occupancy. This is particularly important for halls where the audience provides the major part of the required sound absorption.

For good visibility, as also for good listening conditions, the successive rows of seats have to be raised over the preceding ones with the result that the floor level rises towards the rear. The rise in level may be between 8 cm to 12 cm per row. As an empirical rule, the angle of elevation of the inclined floor in an auditorium should not be less than 8 degrees.

Where *balcony* is provided, its projection into the hall should not be more than twice the free height of opening of balcony recess. The elevation of balcony seats should be such that line of sight is not inclined more than 30 degrees to the horizontal. If balconies are too deep, *sound shadow* usually occur since the seats underneath the balcony do not receive ceiling reflections. The defect, however, can be rectified by providing reflectors, as shown by dotted lines in Fig. 28.6.

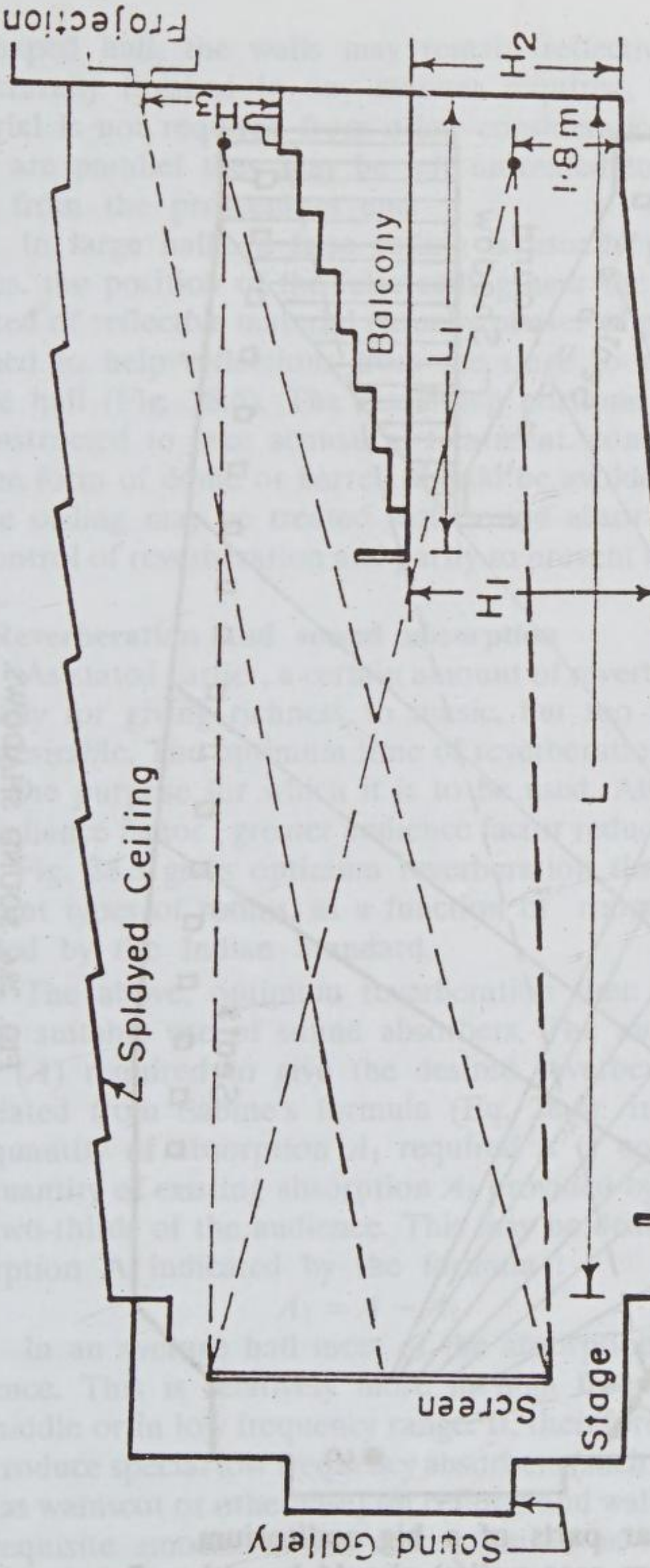


FIG. 28.5. LONGITUDINAL SECTION OF A TYPICAL AUDITORIUM OR A CINEMA THEATRE.
$\alpha \approx 8°$; $L_1 \geq 2H_1$; $L_1 \geq L/3$; $H_2 \geq 3$ m; $H_3 \geq 2.3$ m

5. Treatment of interior surfaces

The treatment of interior surfaces, *i.e.*, ceilings side walls, rear walls play an important role in the acoustical design. The ceilings and walls should provide favourable reflections to reinforce the sound

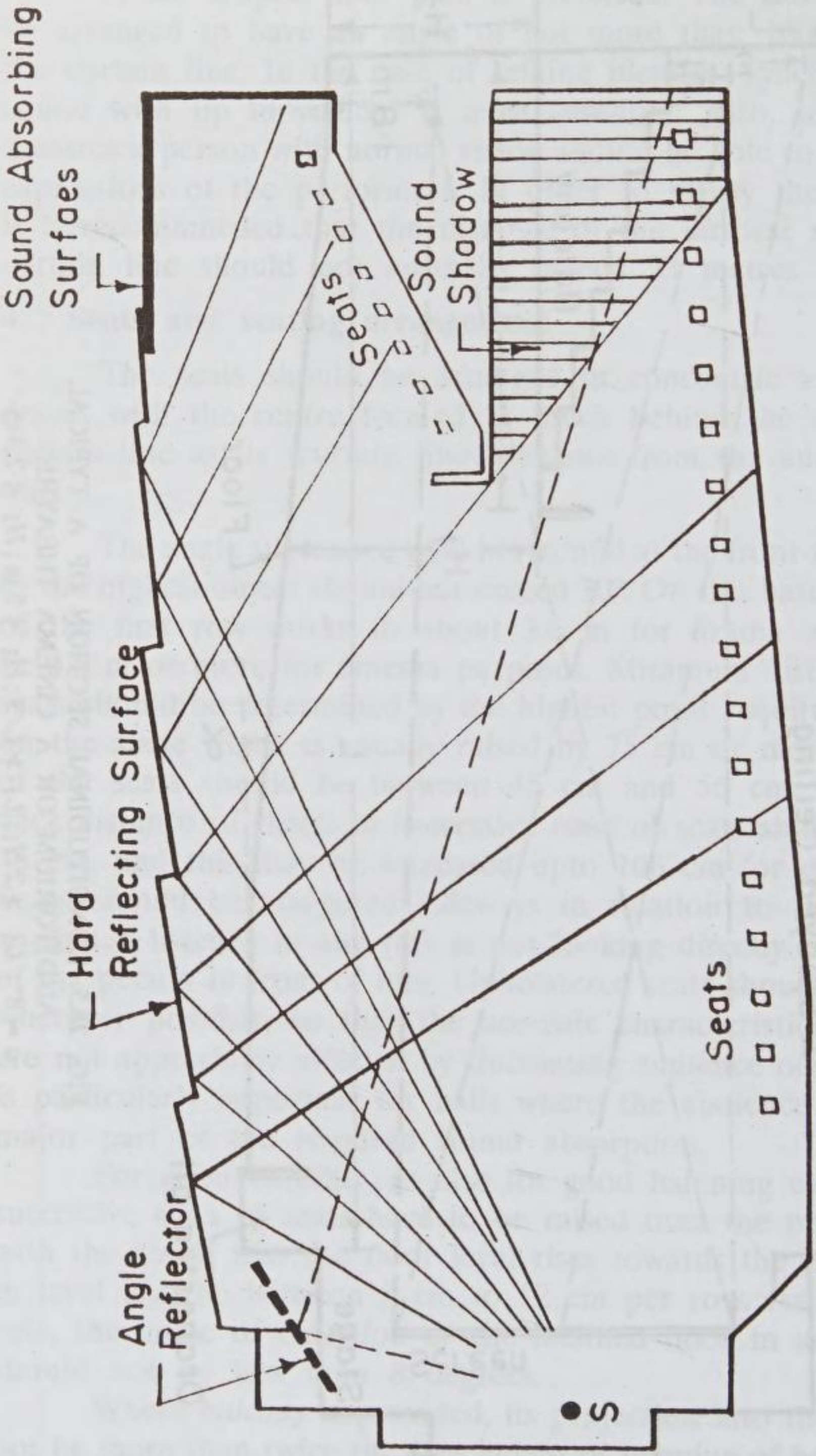


FIG. 28.6. SOUND REFLECTION AND SHADOWS.

that reaches the rear parts of a big auditorium.

The auditorium rear wall(s) should be either flat or convex in shape. This should not be concave in shape, but where it cannot be avoided, the acoustical design should indicate either the surface to be splayed or convex corrugations given in order to avoid any tendency for the sound to focus into the hall.

Where the side walls are non-parallel as in the case of a

fan-shaped hall, the walls may remain reflective and may be architecturally finished in any manner required, if sound absorbing material is not required from other considerations. Where the side walls are parallel they may be left untreated to a length of about 7.5m from the proscenium end.

In large halls, a false ceiling is usually provided below the trusses. the position of the false ceiling near the proscenium is constructed of reflective material (usually plaster of paris) and is suitably inclined to help reflections from the stage to reach the rear seats of the hall (Fig. 28.6). The remaining portions of the false ceiling is constructed to take acoustical treatment. concave shaped ceilings (in the form of dome or barrel) should be avoided. The rear portion of the ceiling may be treated with sound absorbing material partly for control of reverberation and partly to prevent build-up of audience noise.

6. Reverberation and sound absorption

As stated earlier, a certain amount of reverberation is desirable, specially for giving richness to music, but too much reverberation is undesirable. The optimum time of reverberation for a hall depends upon the purpose for which it is to be used. Also, it depends upon the audience factor ; greater audience factor reduces the reverberation time. Fig. 28.3 gives optimum reverberation time at 500 cycles for different types of rooms, as a function of room volume, as recommended by the Indian Standard.

The above, optimum reverberation time has to be adjusted by the suitable use of sound absorbers. The number of absorption units (A) required to give the desired reverberation time may be calculated from Sabine's formula (Eq. 28.4). In order to estimate the quantity of absorption A_1 required it is necessary to calculate the quantity of existing absorption A_2 provided by various furnishings and two-thirds of the audience. This may be deducted from the total absorption A indicated by the formula :

$$A_1 = A - A_2$$

In an average hall most of the absorption is provided by the audience. This is relatively more in high frequency range than in the middle or in low frequency range. It, therefore, becomes desirable to introduce special low frequency absorbers (such as wooden panelling used as wainscot or otherwise) on ceilings and walls which will provide the requisite amount of the absorption so as to achieve optimum reverberation time over as wide a frequency range as possible. The amount of the absorptive materials required should be calculated on the basis of the absorption values of one or more frequencies in each of the low, middle and high frequency regions, namely 125, 500 and 2000 c/s. Upholstered seats should be provided, wherever possible, so that the acoustic characteristics of the hall are not ap-

preciably affected by the fluctuating audience occupancy. This is particularly important for halls where the audience provides the major part of the required sound absorption.

Reflection surfaces should be so designed as to aid distribution of sound. Those areas which cause objectionable sound reflection and need to be treated with sound absorbents should be earmarked for treatment with sound absorbing material. These areas are (*a*) the rear walls, (*b*) the balcony parapet, (*c*) any area which may reflect sound back to stage, (*d*) concave areas which have a tendency to focus sound in certain places, and (*e*) such other areas as will contribute to indirect sound arriving at any point in the hall later than 50 milli-second after the direct sound. The rest of the sound absorbing materials required to be introduced in the room should be distributed over the various remaining surfaces.

28.9. ACOUSTICS OF STUDIOS

A *studio* is a big room or a hall where sound is picked up by a microphone, and is either recorded or broadcast. It includes radio-broadcasting station, television station and sound- recording studio. The basic requirements of such a studio are : (*i*) perfect sound proofing, and (*ii*) variable reverberation time, due to variable pitch and frequency of sound produced there.

The following points are noteworthy for the acoustic design of a studio.

1. The studio walls should be of rigid construction so as to completely insulate and exclude the external noise.

2. The studio should be rectangnlar in plan with ratio of height, breadth and length as 2 : 3 : 5. The ceiling should be flat.

3. The outer surfaces of wall should be of reflective type, while the interior surfaces of walls, ceilings, floors etc. should be of absorbent materials.

4. The noise level in the studio should be brought down to 20 to 30 dB.

5. Provision of windows should be minimum, to prevent transfer of noise from outside.

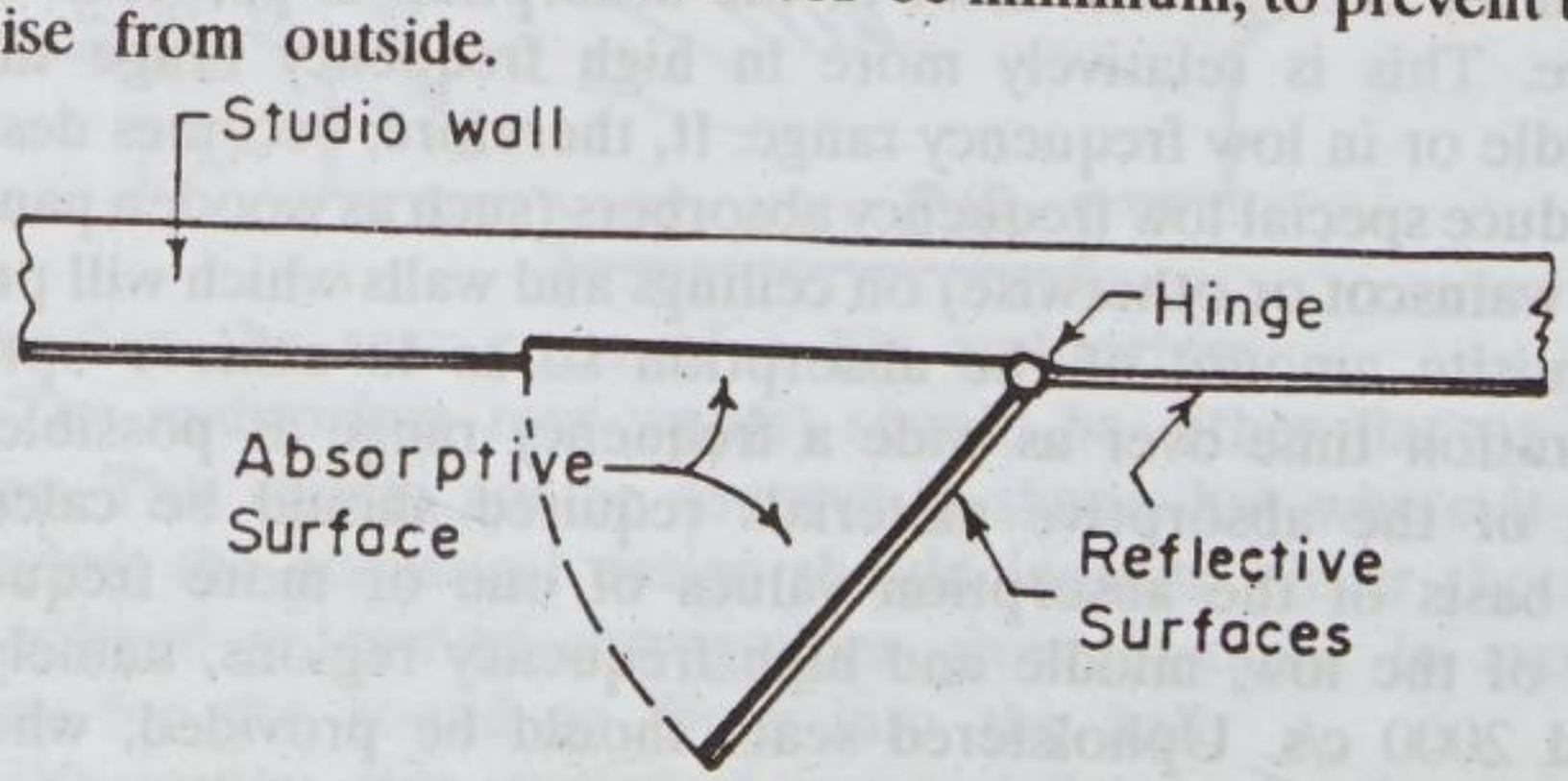


FIG. 28.7. HINGED PANEL.

6. Air-conditioning machinery etc. should be completely isolated, and their noise should be completely insulated.

7. If there are more than one studios in a building ; they should preferably be on the same floor. In no case should two studios be located one above the other ; there should be a gap of atleast one floor.

8. The acoustic design of the studio should be such that echoes and near echoes are completely eliminated

9. Heavy curtains and draperies should be used with advantage to control or regulate the time of reverberation.

10. Variable reverberation time can be obtained by providing *hinged panels* or *shutters*, with one surface of rotatable panel of absorptive material and the other of reflective material (Fig. 28.7). Panels with hinge at the centre may be also used, having two different absorbent materials on both the faces.

11. Reverberation time can also be *varied* by providing rotable cylinders in the ceiling of the studio. Each cylinder or drum (Fig. 28.8) has three sectors, provided with three different absorptive materials. The cylinders can be rotated by rack and pinion arrangement, thus getting the required units of absorption for the desired reverberation time.

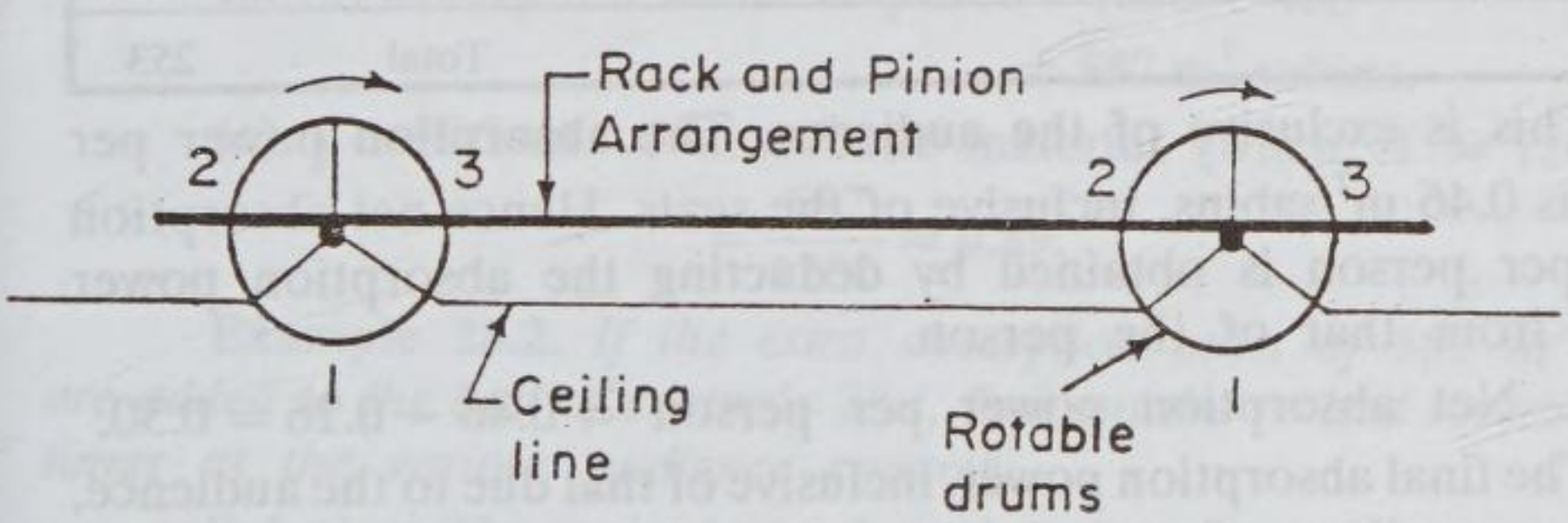


FIG. 28.8. ROTABLE CYLINDERS.

Example 28.1. *An auditorium, rectangular in shape, has the following dimensions : Length = 35 m, breadth = 25 m, and height = 9 m. The internal areas of different surfaces are as follows :*

Cement Plaster : 800 m^2
Concrete Floor : 700 m^2
Timber Floor : 200 m^2
Plaster of pairs ceiling (suspended from truss) : 600m^2
Curtains (light) : 120m^2

The capacity of the auditorium is 1050 seats (chairs, upholstere seat with spring). Determine the following :

(a) Number of absorbing units and time of reverberation when there is (i) no audience (ii) one third audience (iii) two-thirds audience

(iv) full audience.

(b) Number of extra absorbing units required so as to get an optimum reverberation time of 1.2 seconds when the strength of the audience is two-thirds of its capacity.

(c) Coefficient of absorbing material, if the area available for fixing absorbing material, is $1200\,m^2$

Solution (*a*) The existing absorption units, exclusive of audience is computed in the following table. The values are at an average frequency of 550 c/s Table 28.5).

Surface	*Area or No.*	*Absorption coefficient per m^2 or per No.*	*Absorption units in m^2-sabins*
1. Cement plaster	800 m^2	0.02	16
2. Concrete Floor	700 m^2	0.03	21
3. Timber Floor	200 m^2	0.09	18
4. Suspended ceiling	600 m^2	0.05	30
5. Seats (upholstered chairs)	1050 m^2	0.16	168
		Total	253

This is exclusive of the audience. The absorption power per person is 0.46 m^2-sabins, inclusive of the seats. Hence net absorption power per person is obtained by deducting the absorption power of seat from that of the person.

$\therefore$ Net absorption power per person $= 0.46 - 0.16 = 0.30$.

The final absorption power, inclusive of that due to the audience, at various audience factor, is tabulated below.

Audience	*Absorption units of audience*	*Absorption units of materials/surface*	*Total absorption units in m^2-sabins*
1. Nil	–	253	253
2. $\frac{1}{3} \times 1050 = 350$	105	253	358
3. $\frac{2}{3} \times 1050 = 700$	210	253	463
4. 1050	315	253	568

Total volume of auditorium $= 35 \times 25 \times 9 = 7875\ m^3$

The reverberation time for different strength of audience can be worked out from the following Sabin's equation,

$$t = \frac{0.16\,V}{A} \text{ seconds.}$$

The values are tabulated below.

Audience	*Total absorption units (A) in m^2-sabins*	*t seconds*
1. Nil	253	4.98
2 $\frac{1}{3}$ capacity.	358	3.52
3 $\frac{2}{3}$ capacity.	463	2.72
4 Full capacity.	568	2.22

It is to be noted that each one of the above reverberation time is higher than the optimum time of 1.2 seconds.

(*b*) For the optimum reverberation time $t = 1.2$ seconds,

$$t = \frac{0.16\,V}{A}$$

or
$$1.2 = \frac{0.16 \times 7875}{A}$$

From which $A = 1050\ m^2$-sabins.

Absorption power of hall for $\frac{2}{3}$ audience capacity

$$= 463\ m^2\text{-sabins.}$$

Extra absorption units required $= 1050 - 463$

$$= \mathbf{587\ m^2\text{-sabins.}}$$

(*c*) Coefficient of absorbent material (when $A = 1200\ m^2$)

$$= \frac{587}{1200} \approx \mathbf{0.49.}$$

Example 28.2. *If the extra absorption units of 587 m^2-sabins are added to the hall of example 28.1, find out the revised reverberation times at the various audience capacity.*

Solution. The revised reverberation time can be computed from the expression $t = \frac{0.16\,V}{A}$, where A is the total absorption units, inclusive of the extra ones ($= 587\ m^2$) added. The computation are arranged in the tabular form below.

Audience	*Existing absorption units (m^2-seconds)*	*Existing Reverberation time (seconds)*	*Total absorption units $= A + 587\ (m^2)$*	*Revised reverberation time (seconds)*
1. Nil	253	4.98	840	1.5
2. $\frac{1}{3}$ capacity	358	3.52	945	1.33
3. $\frac{2}{3}$ capacaity	463	2.72	1050	1.20
4. Fully capacity	568	2.22	1155	1.09

Example 28.3. *A studio has the following inner dimensions : Length = 12 m, width = 8 m and height = 4.5 m. The ceiling is provided with acoustic tiles having absorption coefficient of 0.35. Curtains in heavy folds, having absorption coefficient of 0.5 is provided on one of the short walls. The absorption power of other surfaces (such as doors, pelmets, etc.) of the studio may be taken as 14.9 m^2-sabins. It is desired to vary the reverberation time from 0.75 to 1 second by the provision of revolving panels with central hinge. Compute the following :*

(a) Extra absorption units required with each time of reverberation.

(b) Coefficients of absorbent materials on both faces of the revolving panel, and

(c) Area of wall panel.

Solution. (*a*) The total available absorption units are tabulated below :

	Surface material	*Area m^2*	*Absorption coefficient per m^2*	*Absorptive units (m^2-sabins)*
1.	Acoustic tiles	96	0.35	33.6
2.	Curtains in heavy-folds	36	0.50	18.0
3.	Other surfaces	–	–	14.9
			Total	6.5 m^2.

Volume of studio $= 12 \times 8 \times 4.5 = 432\ m^2$

(*i*) For $t = 0.75$ second, we have

$$A = \frac{0.16\,V}{t} = \frac{0.16 \times 432}{0.75}$$

$$= 92.16\ m^2.$$

$\therefore$ Extra absorption units required

$$= 92.16 - 66.5 = \mathbf{25.66\ m^2}$$

(*ii*) For $t = 1$ second,

$$A = \frac{0.16 \times 432}{1} = 69.12\ m^2$$

$\therefore$ Extra absorption units required

$$= 69.12 - 66.5 = \mathbf{2.62\ m^2}$$

(*b*) From part (*a*) above, we find that the ratio of the coefficients of absorption of the two materials should be

$$= \frac{25.66}{2.62} = 9.8 \approx 10$$

Hence provide 18 mm thick compressed wood particle board (perforated, with rigid backing), having coefficient of absorption equal to 0.6 on one face of the panel, simple wood panelling, having coefficient = 0.06 on the other face.

(*c*) Area of revolving panel

$$= \frac{25.66}{0.6} = 42.77 \text{ m}^2$$

or $$\frac{2.62}{0.06} = 43.67 \text{ m}^2.$$

Hence provide the revolving panel with central hinge, having an average area of **43 m²**

28.10. SOUND INSULATION

Sound insulation is the measure by which transmission of sound/noise from inside to outside or (*vice-versa*) or from one room to the other is prevented.

Unwanted sound reaching the ears is called *noise*. It may be due to frequency of sound or intensity of sound or both. Noise due to high frequency sound is more unpleasant than the noise due to low frequency sounds. Noisy conditions not only result in uncomfortable living conditions, fatigue, inefficiency and mental strain, but prolonged exposure to such conditions may cause temporary deafness or nervous breakdown.

Following are the *effects* of noise :

(*i*) It creates discomfort.

(*ii*) It has adverse effect on blood pressure, muscular strain and on sleep.

(*iii*) It leads to fatigue, and decreases the efficiency of persons.

(*iv*) It takes away essence of music and speech.

(*v*) It disturbs concentration.

(*vi*) Prolonged exposure to noise may result in temporary deafness or even nervous breakdown.

(*vii*) Reduction in noise increases output of labour.

Noise Classification

From the *origin* point of view, noises may be of two types:

(*a*) Outdoor noises. (*b*) Indoor noises.

Outdoor noises are caused by road traffic, railways, aeroplanes, lifts, moving machinery, machines in nearby factory or building etc. *Indoor noises* are those which are caused either in the same room or in the adjacent room. These are due to conversation of people, moving of people or furniture, crying of babies, playing of radios or other musical instruments, operations of cisterns and water closets, noise of type writer, banging of doors etc.

Noise may be alternatively classified as follows :

(*a*) Air borne noises or sounds.

(*b*) Structure borne noises or impact noises or sounds.

Air borne sounds are those which are generated in air and which is transmitted in air directly to human ear. Such a sound

travels from one part of the building to the other, or from outside of the building to inside by (*i*) openings such as doors, windows, ventilators, key holes etc., or by (*ii*) forced vibrations set up in walls, ceilings etc. Air borne noises possess less power continues for a long duration and is confined to places near its origin.

Structure borne sounds or *impact sounds* are those which originate and progress on the building structure. These are caused by structural vibrations originated due to impact. The common sources of this sound are : foot steps, movement of furniture, dropping of utensils on floor, hammering, drilling, operation of machinery, etc. These are more powerful, propagate over long distances and persist for a very short duration.

The difference between the air borne noise and impact noise is related to the origin of noise in relation to the reciever room only. In a three storey building, washing of clothes in the room of the middle floor will be heard as impact sound for the room below it and as air borne sound in the room above it.

Transmission of noise

Noise is transmitted in the following ways :

(*i*) Through air.

(*ii*) By vibrations of structural members.

(*iii*) Through structural members.

Transmission of noise/sound through air is more common. In this sound waves travel through openings of doors, windows, ventilators, key holes, cracks in the walls etc. When the source of sound is very near, sound waves impinge or strike on the thin structural members such as doors, partition walls, membrane walls etc. These structural members vibrate and in turn set up secondary sound waves to the other side. The third type of transmission takes place when elastic wave motions, consisting of compression and rarefactions of sound, are transmissted from particle to particle of the structural member, in the form of pressure impulses. Such a mode is prevalent where mechanical vibrations are caused, such as in factories, workshops, etc.

Transmission losses

When sound is transmitted from the source or origin to the adjoining room/area, reduction in sound intensity takes place. This is known as *transmission loss* (TL). It is numerically equivalent to the loss in the intensity of sound expressed in decibels. If 60 dB and 40 dB are the sound levels measured on either side of a wall, the transmission loss= 60 – 40 = 20 dB. The following points are noteworthy in connection with transmission losses.

1. Transmission loss is expressed in terms of loss of sound intensity (expressed in decibels).

2. The efficiency of *sound insulation* of barrier (such as wall,

partition, door, floor, etc.) is expressed in terms transmission loss of air-borne sound passing through the barrier.

3. The transmission loss (or sound insulation) offered by a structure depends upon the materials used and method of construction.

4. Transmission loss depends upon the frequency of sound. Hence transmission loss of a structure should be studied over a wide range of sound frequencies.

Acceptable indoor noise levels

Acceptable noise levels are those which will neither cause uncomfortable conditions nor damage the acoustics of the building. Acceptable noise levels depend upon (*a*) nature and type of noise, (*b*) time of fluctuations of noise, (*c*) background noise, and (*d*) type and use of building. The generally acceptable noise levels inside buildings from point of view of comfort, economy and practical considerations under conditions prevailing in our country are given in Table 28.8.

TABLE 28.8. ACCEPTABLE INDOOR NOISE LEVELS

S. No.	*Type of building*	*Noise level (dB)*
1.	Radio and TV studious	25—30
2.	Music room	30—35
3.	Hospital and auditoria	35—40
4.	Apartments, hotels, and homes	35—40
5.	Conference rooms, small offices and libraries	35—40
6.	Court rooms and class rooms	40—45
7.	Large public offices, banks and stores	45—50
8.	Restauratns	50—55
9.	Factories	55—65

Sound insulation

Sound insulation or *sound proofing* is a measure used to reduce the level of sound when it passes through the insulating building component. It has different function than sound absorption. The function of a sound-absorbent material is to absorb and thus reduce the sound reflected from a surface, while the function of sound insulating construction is to reduce sound *passing* through it. Sound absorbers, mostly of porous materials, are poor sound insulators, while hard materials, used for sound insulation are poor absorbers.

The material and methods used for sound insulation should be such that desirable insulation is obtained. Table 28.9 gives the desirable levels of sound insulation between individual rooms (air borne) :

TABLE 28.9.
SOUND INSULATION BETWEEN INDIVIDUAL ROOMS

S.No.	*Situation*	*Overall Insulation in dB*
1.	Between living room in one house or flat and the living room and bed rooms in another	50
2.	Elsewhere between houses or flats	40
3.	Between one room and another in the same house or flat	30
4.	Between teaching rooms in a school	40
5.	Between one room and another room in an office	30
6.	Between one ward and another ward in a hospital (*a*) Normal wards (*b*) Extra -quiet special wards	 40 45

The floor of a room immediately above the bedroom or living room, should have *impact insulation* as indicated below :

(*i*) *Concrete floors*. In the case of houses and flats, these floors should be insulated so as to reduce the average loudness of impact sound by about 15 dB above that provided by a bare concrete floors of a normal thickness.

(*ii*) *Timber floors*. In the case of houses and flats, these floors should be insulated so as to reduce the average loudness of impact sound by about 20 dB as compared with the normal floor construction of a wooden board joint floor with lath and plaster ceiling.

Reduction of impact noise requires the use of discontinuous or non-homogeneous materials in the construction of the structure.

Sound Insulating Materials

(a) Non-porous rigid partitions

The sound insulation of non-porous, homogeneous rigid constructions, such as plastered solid brick masonry walls varies as the logarithms of the weight per unit area (Table 28.10). There is thus, a limit beyond which it requires excessive increase on thickness to produce only a small increase in sound insulation. The relation between the weight of a rigid partition and its sound insulation is approximately such that every time the weight is doubled there in an increase in sound insulation of about 4 to 5 dB.

TABLE 28.10. VARIATION OF SOUND INSULATION VALUES WITH WEIGHT

Weight per m^2 of wall area (kg)	*Average TL dB*	*Weight per m^2 of wall area (kg)*	*Average TL dB*
5	22.8	350	50
25	33.2	400	50.9
50	37.6	450	51.6
100	42.0	500	52.3
200	46.4	550	52.9
250	47.9	600	53.6
300	49.1		

(b) Porous rigid materials

The values given in table 28.10 do not apply in the case of porous rigid materials such as porous concrete masonry, cinder concrete etc., because of their sound absorptive properties which provide about 10 percent higher insulation than the non-porous variety of the same weight. In order to secure the best insulation from such porous materials, it is recommended that porous partitions should be plastered at least on one side,and if possible on both sides.

(c) Non-rigid or flexible porous materials

These consists of felt, mineral wool and quilt etc. They by themselves provide low sound insulation as compared to rigid materials and therefore they are not generally used for the purpose of noise reduction and isolation. However, a composite construction employing a combination of rigid materials and porous absorbers may be adopted where weight is an important factor. Such a construction produces better insulation per unit weight of parition.

Wall Insulation : Vertical barriers

Walls and partitions are the vertical barriers to noise. Their proper design and construction may insulate the sound to the desired level. Wall construction, used for sound insulation, may be of three types : Rigid and massive homogeneous walls, (*b*) Partitions of porous materials, (*c*) Double wall partition, and (*d*) Cavity wall type construction.

(a) Rigid homogeneous walls

A rigid wall consists of stone, brick or concrete masonry construction, well plastered on one or both the sides. The sound insulation offered by these rigid walls depends upon their weight per unit area, as indicated in Table 28.10. The sound insulation thus increases

with the increase in the thickness of the wall. Because of the logarithmic variation between weight and transmission loss, such a construction (*i.e.* solid wall) becomes highly uneconomical and bulky after certain limit. As can be seen from Table 28.11, the transmission loss (sound insulation) of a one-brick wall is 50 dB while that of a $1\frac{1}{2}$ brick thick wall is 53. On the other hand, a half-brick wall with 13 mm thick plaster on both the sides as TL of 45 dB.

(b) Partition walls of porous materials

As indicated earlier, porous materials may be rigid or non-rigid. Rigid porous materials (such as porous concrete masonry, cinder concrete etc.), the insulation increases about 10% higher than the non-porous rigid material. However, partition walls of non-rigid porous materials (such as felt, mineral wool etc.) offer very low sound insulation, though they can be used in combination with rigid materials with added advantage.

(c) Double wall partition

A double wall partition, shown in Fig. 28.9, consists of plaster boards or fibre boards or plaster on laths on both the faces, with sound absorbing blanket in between. Staggered wooden studs are provided as support, though their number should be a minimum. A double wall construction is thus a partition wall of rigid and non-rigid porous materials.

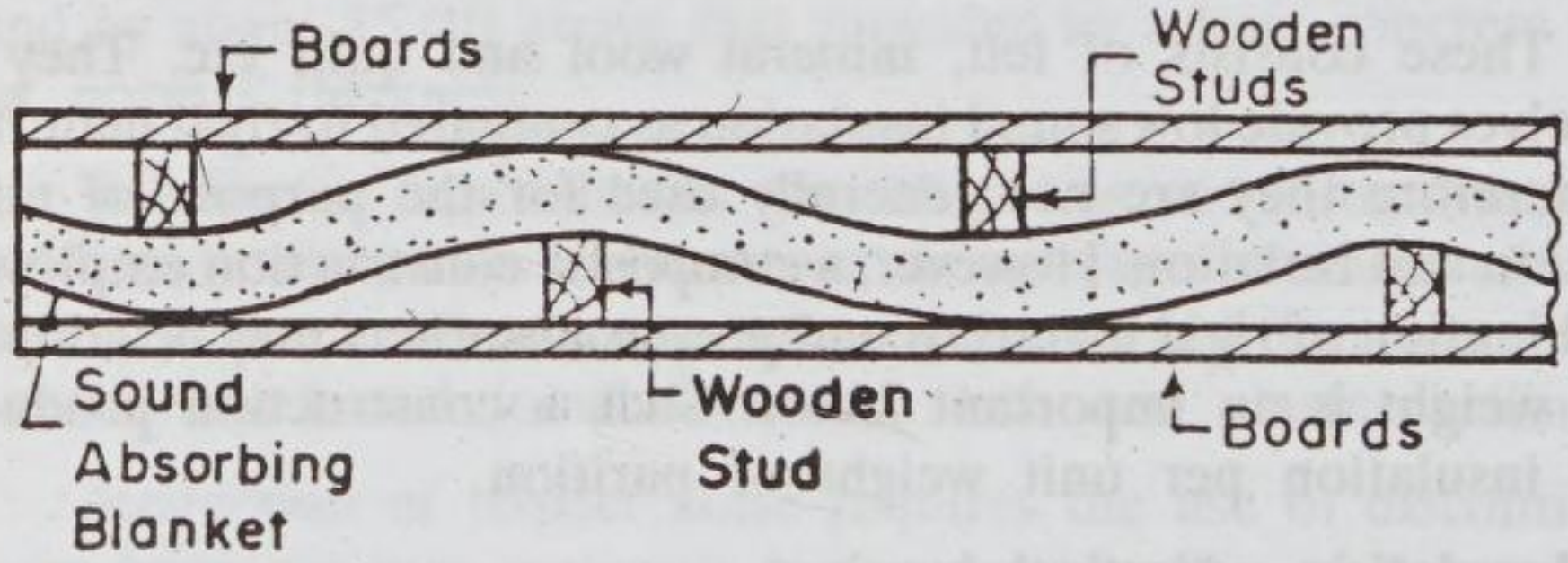


FIG. 28.9. DOUBLE WALL PARTITION.

(d) Cavity wall construction

This is an ideal construction from the point of view of sound proofing, as shown in Fig. 28.10. The gap between the two leafs of the wall may be left air-filled or else filled with some resilient material, like quilt etc., well suspended in the gap. The two faces of the wall may be fixed with celotex or other insulating board. The width of cavity should be atleast 5 cm, and the two wall leaves should be tied by use of only light butterfly wall ties.

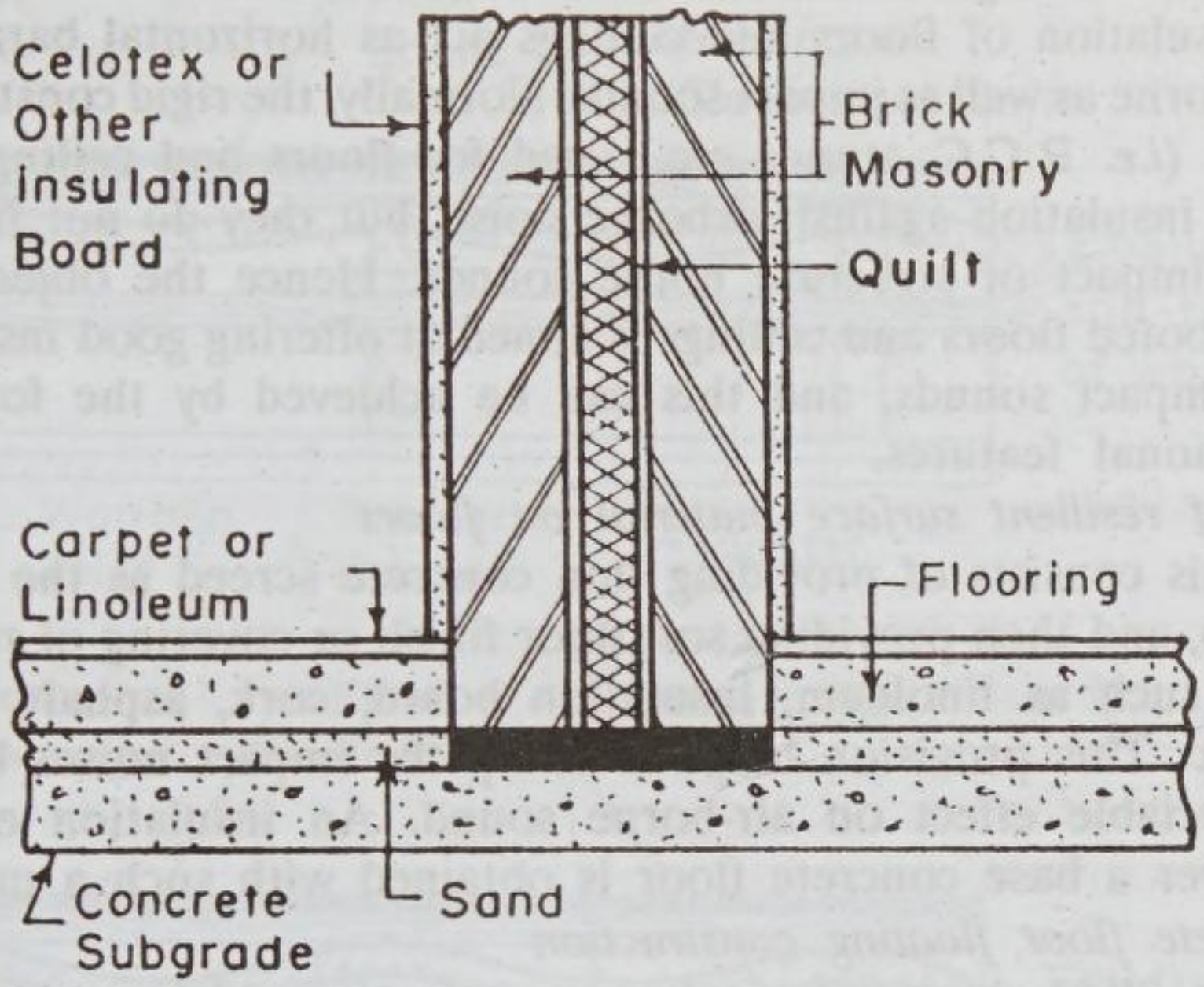


FIG. 28.10 CAVITY WALL OR DOUBLE WALL CONSTRUCTION.

Table 28.11 gives typical insulation values of various types of walls.

TABLE 28.11
TYPICAL INSULATION VALUES FOR DIFFERENT TYPES OF WALLS

Type of construction	*Approx. weight in kg/m²*	*Average TL in dB*
1. One brick thick (*i.e.* 20 cm) wall.	485–490	50
2. One and a half brick thick (*i.e.* 30 cm) wall	705–710	53
3. Cavity wall having two leaves each of half brick thickness (*i.e.* 10 cm) with 5 cm cavity.	485–490	50–53
4. Cavity wall having two 10 cm thick leaves of clinker block with 5 cm cavity.	310–312	50
5. Half brick wall with 13 mm thick plaster on both sides.	268–270	45
6. 20 cm thick hollow dense concrete block wall with 13 mm thick plaster in both sides.	185	45
7. Partition wall made with gypsum wall board fixed on timber frame work.	68–70	45
8. 76 mm thick hollw clay block wall with 13 mm thick plaster on both sides.	108–110	36

Floors and ceilings insulation : Horizontal barriers

Insulation of floors and ceilings act as horizontal barriers to both airborne as well as impact sounds. Normally, the rigid construction materials (*i.e.* R.C.C, stones etc.) used for floors and ceilings offer excellent insulation against airborne noise, but they do not function well for impact or structure borne sounds. Hence the objective of sound-proofed floors and ceilings is aimed at offering good insulation against impact sounds, and this can be achieved by the following constructional features.

1. Use of resilient surface material on floors

This consists of providing thin concrete screed as the R.C.C. floor slab, and then providing soft floor finish or covering of resilient material such as linoleum, insulation board, cork, asphalt mastic, carpet etc. This provision helps to damp the impact noises but has no appreciable effect on air-borne sound. An insulation of 5 to 10 dB over a base concrete floor is obtained with such a material.

2. Concrete floor floating construction

Fig. 28.11 shows a typical construction, in which an additional floor is constructed and isolated (or floated) from the existing concrete floor. Resilient material like quilted mineral or glass wool is laid over the R.C.C. floor/roof. A water proof paper is then laid over it, and then 5 cm thick concrete screed is provided. It is important that both the quilt and the water-proof paper are lapped so as to prevent concrete from getting through. Such a construction provides good insulation against impact sounds.

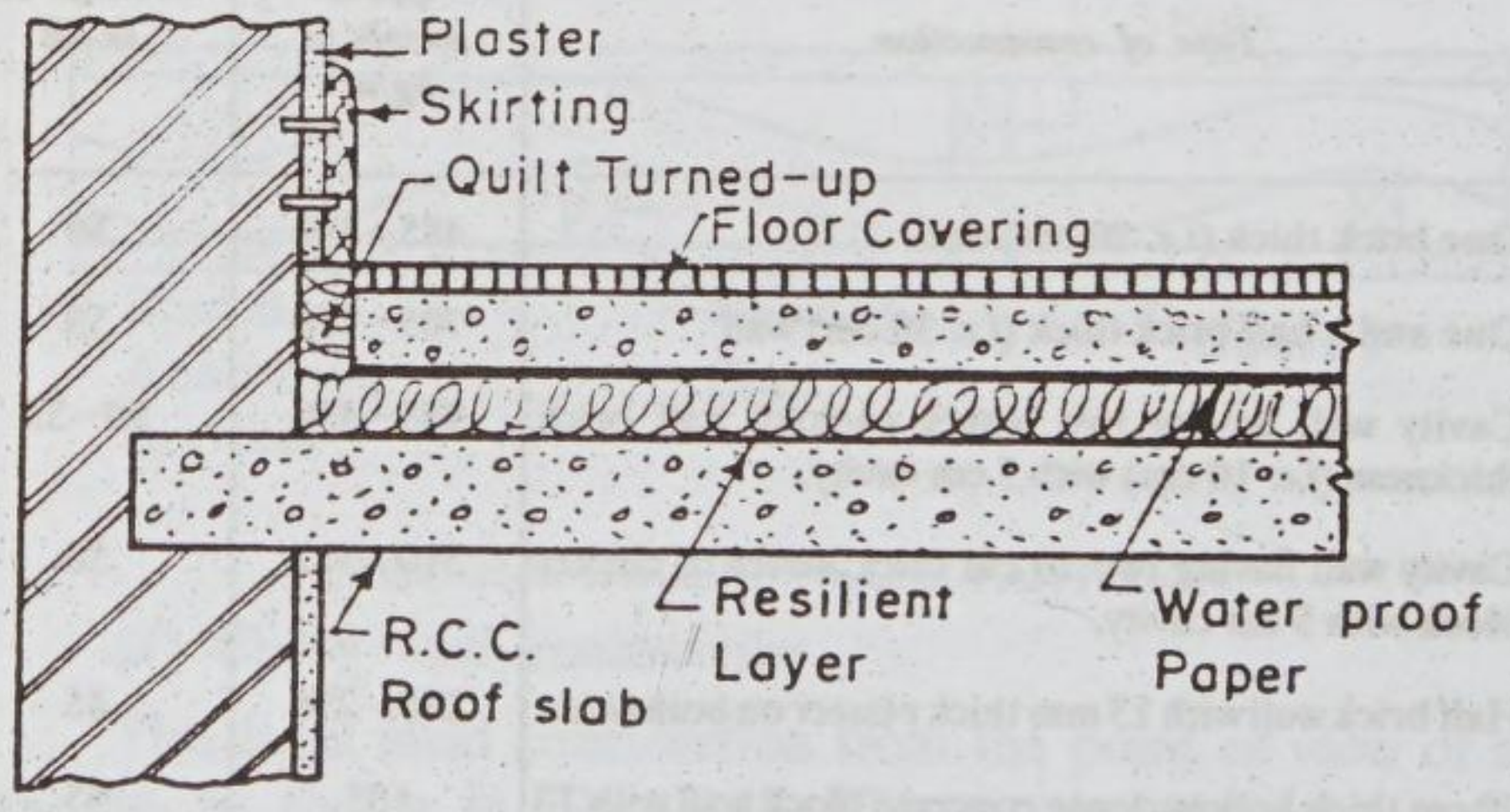


FIG. 28.11. CONCRETE FLOOR FLOATING CONSTRUCTION.

3. Timber floor floating construction

In case of floors constructed of wooden joists, the problem of sound insulation becomes more difficult particularly in the presence of heavy mechanical impact sounds. Fig. 28.12 shows the methods of insulating such existing floors while Fig. 28.13 shows new timber

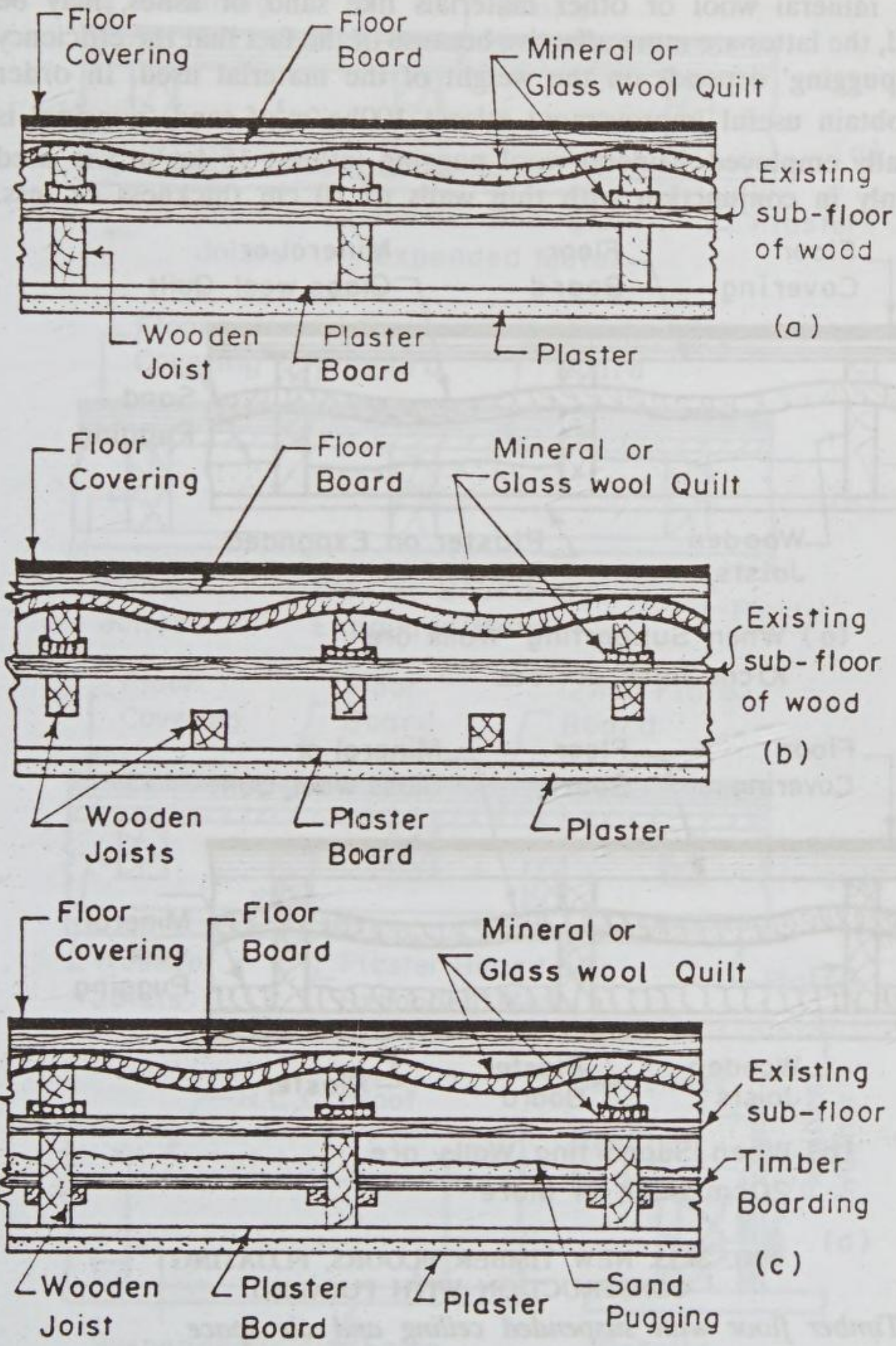
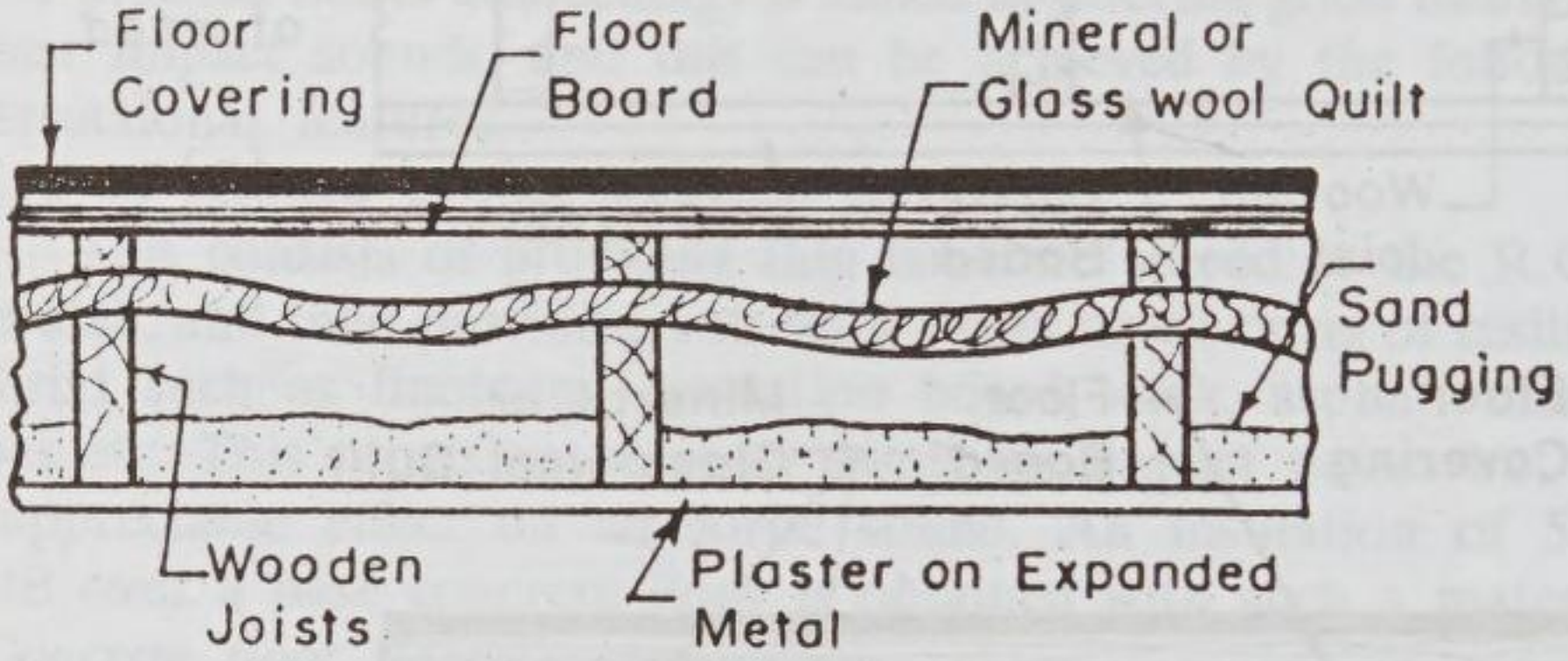


FIG. 28.12. EXISTING TIMBER FLOORS, FLOATING CONSTRUCTION.

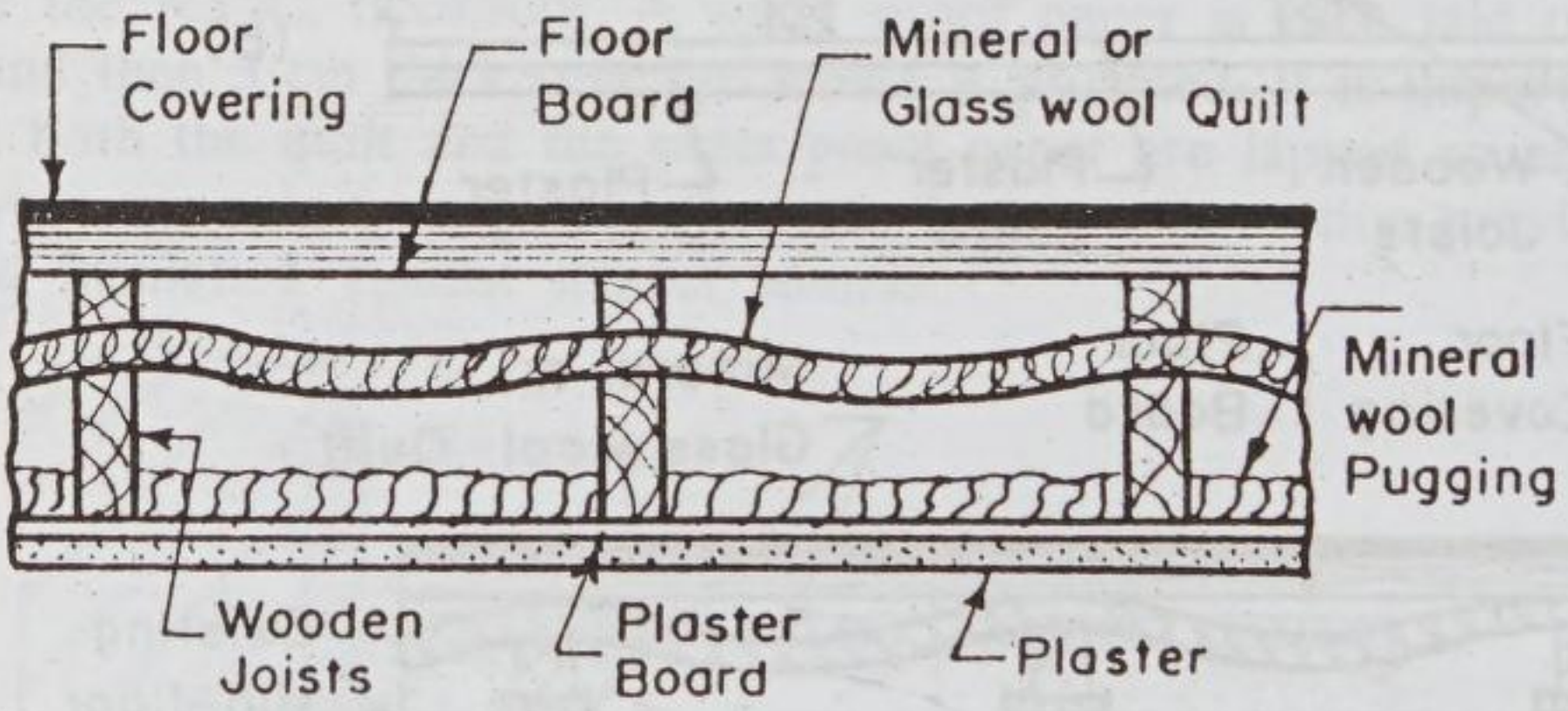
floors, employing mineral or glass wool quilt for isolation purposes. Resilients mountings may be used to obtain even more satisfactory results.

A further improvement in the insulation of such floors is achieved by employing a *pugging* or *deadening* material in the air space between the wood joists. Either sound absorbent type materials

like mineral wool or other materials like sand or ashes, may be used, the latter are more effective because of the fact that the efficiency of 'pugging' depends on the weight of the material used. In order to obtain useful improvement atleast 100kg/$m^2$of sand 'pugging' is usually employed. Mineral wool pugging (atleast 15 kg/m^2) is used mainly in conjunction with thin walls of 10 cm thickness or less.



(a) When Supporting Walls are 10cm thick or less



(b) When Supporting Walls are 20cm thick or More

FIG. 28.13. NEW TIMBER FLOORS, FLOATING CONSTRUCTION WITH PUGGING.

4. Timber floor with suspended ceiling and air space

This type of construction helps to improve the insulation of both air-borne and structure-borne sounds by attenuating and isolating them from room below. Fig. 28.14 (*a*), (*b*) and (*c*) shows typical constructions, with increasing degree of insulation. For solid floors, metal hangers of acoustic clips may be used to support the ceiling below, as shown in Fig. 28.14 (*d*). The extent of improvement effected depend upon the weight of the ceiling as well as on the structural

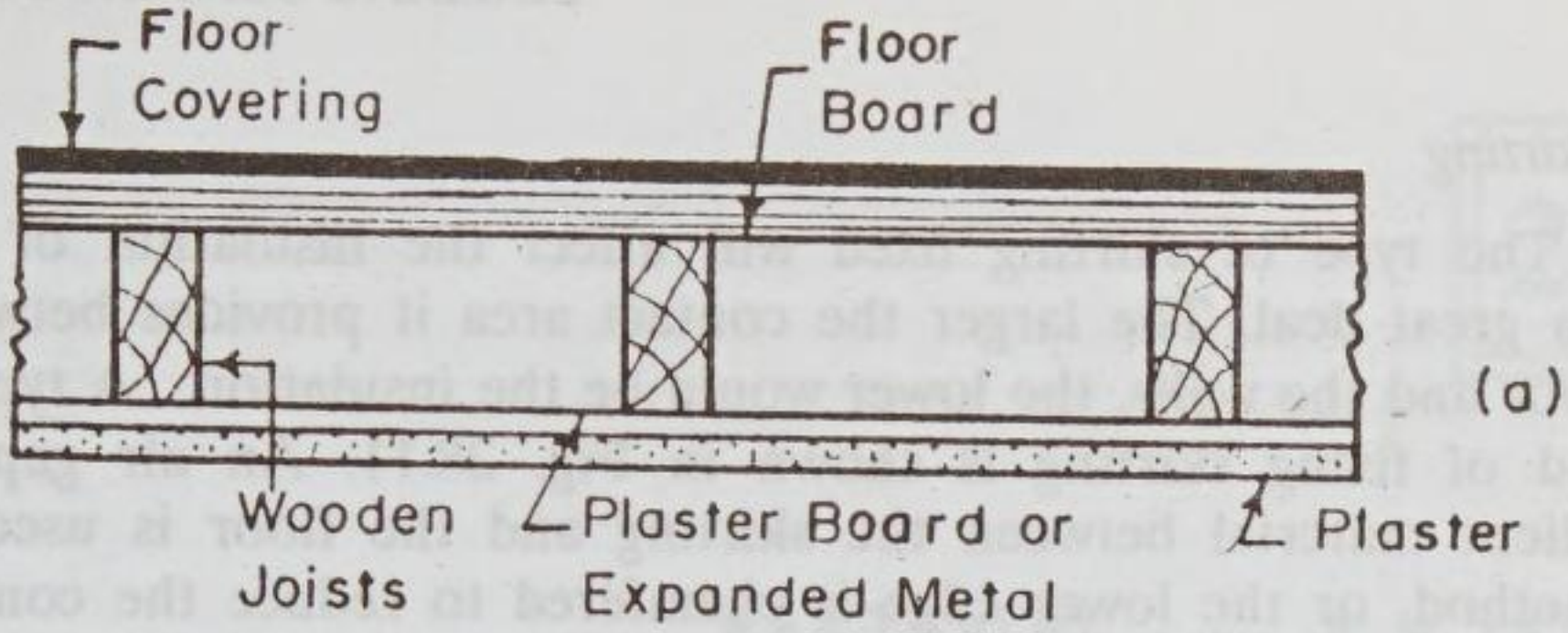


FIG. 28.14. TIMBER FLOORS WITH SUSPENDING CEILING.

rigidity with which it is connected to the solid or wooden floor. Thus the highest insulation could be achieved by using a very heavy ceiling which is arranged to be independent of the floor by supporting it on resilient mountings.

5. Skirting

The type of skirting fixed will affect the insulation of the floor a great deal. The larger the contact area it provides between the floor and the walls, the lower would be the insulation. A typical method of fixing skirting is shown in Fig. 28.11. An air gap or a resilient material between the skirting and the floor is used in this method, or the lower edge is chamfered to reduce the contact area. Again, most of the inner portion of the skirting is scooped out to minimize contact with the walls.

PROBLEMS

1. Explain in brief various characteristics of sound, as applicable to the acoustic design of a room.

2. Write short notes on (*a*) reflection of sound and reflecting surfaces (*b*) echo (*c*) reverberation time.

3. What do you understand by reverberation ? How do you determine reverberation time ? Specify optimum reverberation time for (*a*) cinema theatres (*b*) large halls (*c*) lecture halls.

4. (*a*) What do you understand by absorption of sound ? How does this take place ? Enumerate various absorptive surfaces.

(*b*) Enumerate various requirements of a good acoustic material.

5. Explain in brief general common acoustic defects, and suggest the remedial measures.

6. Discuss in brief general principles and factors in acoustical design of a hall.

7. Write a note on acoustics of studios.

8. Explain how do you determine the absorption units of a cinema theatre/auditorium.

9. Write a note on 'noise'. What are its effects.

10. Explain various methods of sound insulationthrough the wall of a hall.

11. How do you insulate ceiling of a room/hall against external sound?

29

PERT and CPM

29.1. PROJECT MANAGEMENT

A project is composed of jobs, activities, functions or tasks that are related one to the other in some manner, and all of these should be completed in order to complete the project. Every project has one specific purpose : it starts at some specific moment and it is finished when its objectives have been fulfilled. For completion of a project, two basic things are required : (*i*) material resources, (*ii*) man power resources. Many countries, rich in material resources are exceedingly poor in terms of level of production or plan achievement, while there are other countries which have very limited natural resources but have achieved higher level of productivity mainly because of talents, skills, experience and know how of their people. Availability, quality and use of human resources is a single determinant factor in accomplishing project objectives.

Rapid accumulation of scientific technique in the recent past has not been matched by a corresponding improvement in the sphere of human group relations. In other words, sociology has not kept pace with technology. We are not in a position to utilize fully our technology advancement unless we are also able to advance in social sphere. Here comes the role of *management*. While *technology* deals with *material things, management* deals with both *material things* as well as *human beings.*

Management increases the productivity through technological innovation taking into account human factors involved in these advances:

Each project, whether big or small has three *objectives* :

(*i*) The project should be completed with a minimum of elapsed time.

(*ii*) It should use available manpower and other resources as sparingly as possible, without delay.

(*iii*) It should be completed with a minimum of capital investment, without delay.

Project management is a highly specialised job, to achieve the above objectives. Project management involves, the following three *phases* :

1. Project *planning*
2. Project *scheduling*
3. Project *controlling*

Out of the above three phases of project management the first two phases are accomplished before the actual project starts. The third phase is operative during the execution of the project, and its aim is to recognize the difficulties during the execution and to apply measures to deal with these difficulties.

29.2. METHODS OF PLANNING AND PROGRAMMING

Complex research and development projects can be managed effectively if project managers have the means to plan and control the schedules and costs of the work required to achieve their technical performance objectives.

When the planning of a project is undertaken a host of questions arise : How should the work be accomplished ? What resources will be needed ? How long will it take ? How much will it cost? The answers to all these questions can be found by following the modern techniques of project management.

Managers at all levels need improved techniques at all stages in a project to :

(*i*) define the work to be performed.

(*ii*) develop more realistic schedule and cost estimates based or resources planned to perform the work.

(*iii*) determine where resources should be applied to best achieve the time, cost and technical performance objectives.

and (*iv*) identify those areas developing potential delays or cost overruns, in time to permit corrective action.

Following are some of the tools or techniques of project management :

(1) Bar charts and Milestone charts.

(2) Net work diagrams.

1. Bar charts and Milestone charts

Bar charts were introduced by Henery Gantt around 1900 A D. Bar charts represent pictorial representation in two dimensions of a project by breaking it down into a number of manageable units or activities for planning and control shown on the dimension or axis and the durations assigned to these activities on the other dimension or axis. Bar charts were later modified to yield the milestone charts.

2.. Network Methods

Network diagram is an outcome of the improvements in the milestone charts. The network technique is a major advance in management science. This technique is based on the basic *characteristics* of all project, that all work must be done in well-defined steps. For example, for completing a foundation the various steps are (*i*) layout (*ii*) digging (*iii*) placing side boards and (*iv*) concreting. The net-work technique exploits this characteristics by representing the steps of the project objective graphically in the form of a network or arrow diagram. It would be difficult to find in the history of management methods any technique which has received such widespread attention as that accorded to network methods for planning, scheduling and controlling.

The network techniques are called by various names such as PERT, CPM, UNETICS, LESS, TOPS and SCANS. However, these and other systems have emerged from the following two major *network systems* : (*i*) PERT (*ii*) CPM.

The other systems by and large differ from their parents only in non-essentials.

(*a*) **PERT.** PERT stands for 'Program Evaluation and Review Technique. The method was basically developed by the Navy Special Projects Office in co-operation with Booz, Allen and Hamilton, a management consulting firm and Lockheed Missile System Division for evaluating the feasibility of existing schedules on Polaris missile program and for reporting progress.

PERT system is preferred for those projects or operations which are of non-repetitive nature or for those projects in which precise time determination for various activities cannot be made. In such projects management cannot be guided by the past experience. They are referred to as *once through* operations or projects.

(*b*) **CPM.** CPM stands for 'Critical Path Method'. In CPM, networks, the whole project consists of a number of clearly recognizable jobs or operations called *activities* . Activities are usually the operations which take time to carry out, and on which resources are expended. CPM networks are generally used for repetitive type projects or for those projects for which fairly accurate estimate of time for completion of each activity can be made.

29.3. BAR CHARTS

Bar charts were introduced by Henery Gantt around 1900 AD. In his work on production control, Gantt developed the now famous Gantt chart still used on many projects of moderate magnitude. A *bar chart* consists of two co-ordinate axes, one (usually horizontal axis) represent the time elapsed and the other (the vertical axis) represent the jobs or activities to be performed. Each bar represent

one specific job or activity of the project. The beginning and end of each bar represent the time of start and time of finish of that activity ; the length of bar therefore, represents the time required for the completion of that job or activity.

Fig. 29.1 shows the bar chart for a project which has seven distinct jobs or activities (*P, Q, R, S, T, U, V*) to be performed for its completion. The time durations required for the completion of these activities are 10, 5, 10, 7, 5, 8 and 15 unit days respectively. From the chart, we conclude the following :

(*i*) Activities *P* and *Q* can start simultaneously, at zero time. Both the activities are independent. However, activity *Q* is completed much earlier than activity *P*.

(*ii*) Activity *R* starts only when activity *Q* is complete.

(*iii*) However, activity *S* is independent of activity *R*. It starts earlier than *R* and is completed earlier.

(*iv*) Activity *T* starts only when activity *S* is complete.

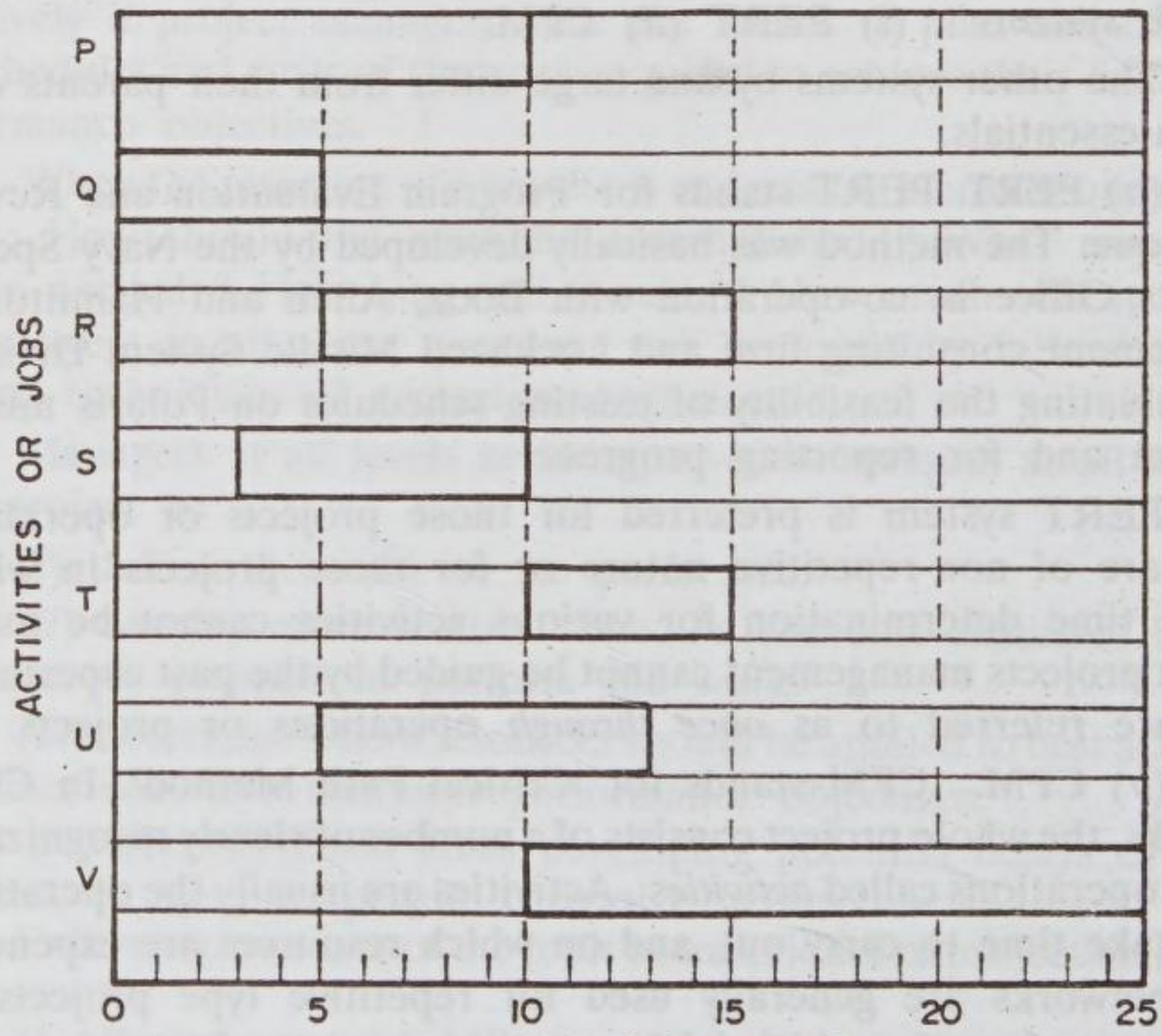


FIG. 29.1. BAR CHART.

(*v*) Activities *U* and *R* can start simultaneously–when activity *Q* is complete.

(*vi*) Activity *V* can start when activities *P* and *S* are complete. End of activity *V* marks the completion of the project.

Each bar of a bar chart can be represented either by a set of two lines running parallel of two each other (*i.e.* by rectangle)

or by a thick Solid line. The first form is preferred.

Example 29.1. *A typical small house construction project consists of the following operations along with the time set for its completion.*

S.No.	*Operation*	*Time (in days)*
1.	*Survey, design and layout*	*3*
2.	*Construction of foundations*	*5*
3.	*Construction of superstructure*	*11*
4.	*Roofing*	*5*
5.	*Fixing doors and window frames*	*2*
6.	*Plumbing and house drainage*	*3*
7.	*Electric fitting*	*3*
8.	*Plastering*	*4*
9.	*Flooring*	*4*
10.	*Carpentary work*	*4*
11.	*Construction of boundary wall and other minor items*	*3*
12.	*Land shaping and clearing*	*2*
13.	*White-washing of walls and painting of doors*	*3*
14.	*Inauguration*	*1*

The project commences on Wednesday, 14th October. Assuming five working days in a week, prepare bar chart of the project. State the assumptions made. Also determine (a) total time, and date of completion of the project (b) expected progress by 10th November.

Solution. The bar chart is shown in Fig. 29.2, prepared with the following assumptions regarding the sequence of various activities:

(*i*) Activity 2 can start only after activity 1 is over.

(*ii*) Activity 3 can start even when half the work of activity 2 is over.

(*iii*) Activity 4 and 5 can start concurrently, but only after activity 3 is over.

(*iv*) Activity 6 and 7 can start concurrently, but only after activity 5 is complete.

(*v*) Activity 8 can start only after activities 6 and 7 are complete.

(*vi*) Activity 9 can start even when half of activity 8 is over.

(*vii*) Activity 10 can start only when activity 9 is over.

(*viii*) Activity 11 can start only when activity 8 is over.

(*ix*) Activity 12 can start only when activity 11 is over.

(*x*) Activity 13 can start even when activity 10 is half over.

(*xi*) Activity 14 is the last activity which marks the completion of the project.

Cross (×) denotes the day on which there will be no construction work.

From the bar chart shown in Fig. 29.2, we find that project will be complete on 30th November–48 days after its start. Also, the progress upto 10th November will be as follows :

(*a*) Activities 1, 2, 3 and 5 will be completely over.

(*b*) Activities 4, 6, 7 and will have 2 days work left.

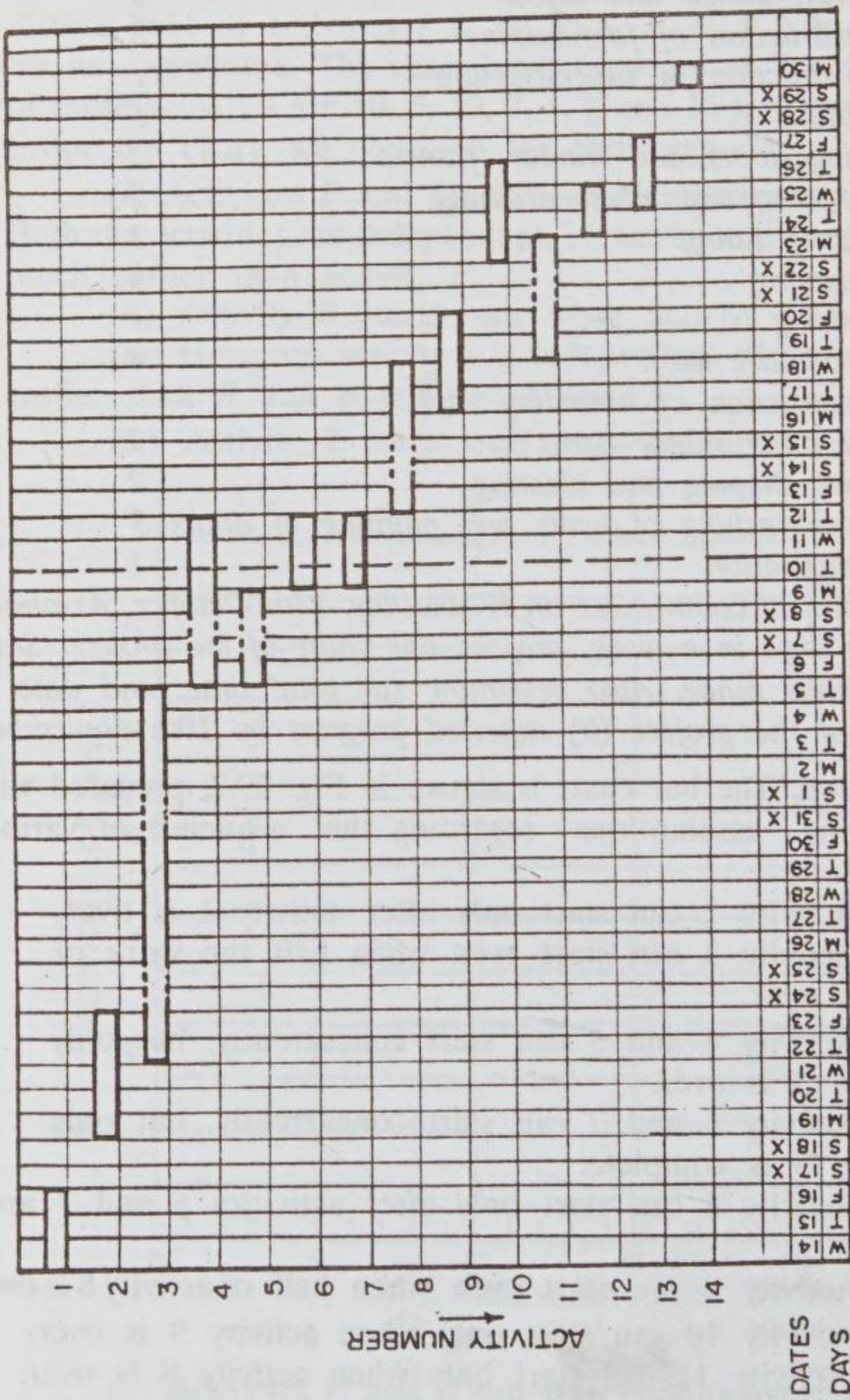


FIG. 29.2. BAR CHART FOR CONSTRUCTION OF A HOUSE.

29.4. SHORTCOMINGS OF BAR CHARTS AND REMEDIAL MEASURES

Bar charts have following short comings. These short comings can be p?.tly overcome by following the suggested remedial measures.

1. Lack of degree of details

On bar chart, only major activities are shown. If too many activities or tasks are separately shown, it becomes clumsy. Due to this, bar charts are not very useful for big projects. A particular activity, whether big or small, is shown by one bar, without any details sub-activities contained in it. These sub-activities cannot be separated out. Due to this, effective control over the activities cannot be achieved.

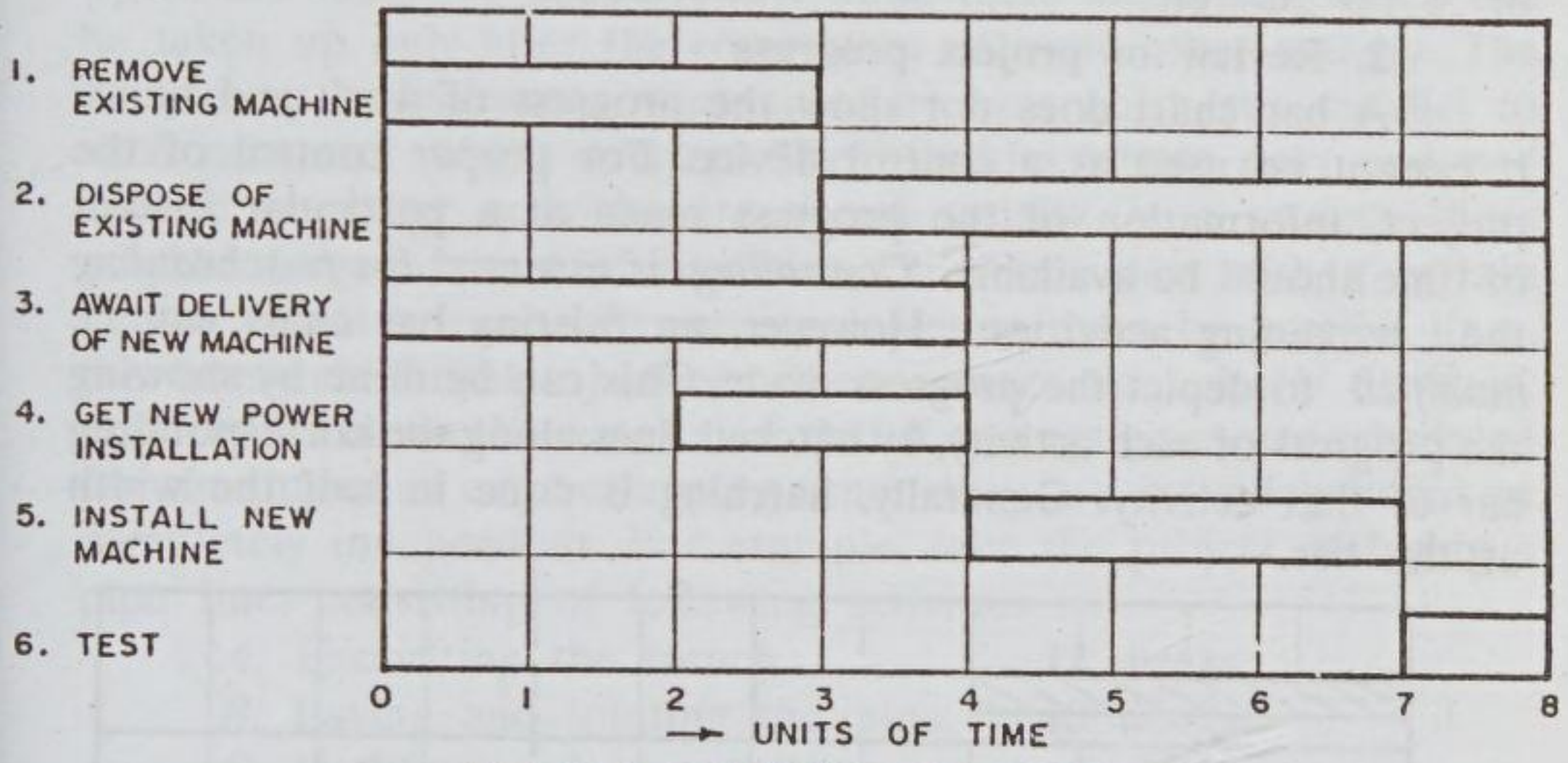

1. REMOVE EXISTING MACHINE
2. DISPOSE OF EXISTING MACHINE
3. AWAIT DELIVERY OF NEW MACHINE
4. GET NEW POWER INSTALLATION
5. INSTALL NEW MACHINE
6. TEST
0 1 2 3 4 5 6 7 8
UNITS OF TIME

FIG. 29.3. ORIGINAL BAR CHART.

For example, consider the activity '*dispose of existing machine*' in the bar chart (Fig. 29.3) prepared for the task of obtaining a new machine. This item will be represented in the chart by one bar through the following functions control the completion time for the activity.

A. Dumping machine in store.
B. Notice inviting bidding
C. Finalisation of highest bid.
D. Final disposal of the machine.

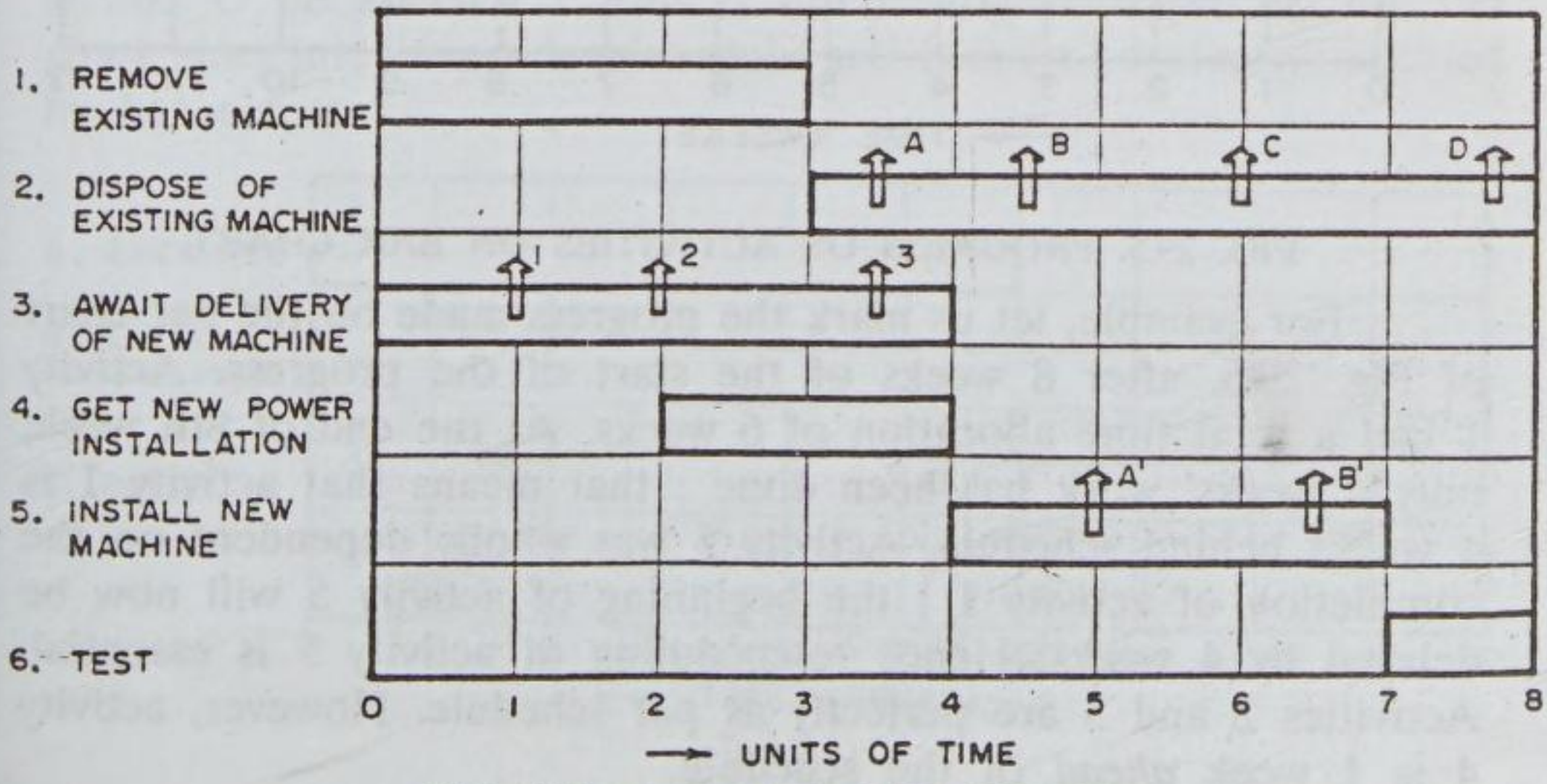


FIG. 29.4 IMPROVED BAR CHART

For effective completion of the main activity, these sub- activities should be scheduled properly. The above information can be shown effectively by marking stages (called *'mile stones'*) on the activity bar, shown in Fig. 29.4. Similarly, sub-activities of other activities of each bar can be marked with stages or mile stones 1, 2, 3 etc. or A' , B' , C' etc.

2. Review of project progress

A bar chart does not show the progress of work and hence it cannot be used as a control device. For proper control of the project, information of the progress made at a particular instant of time should be available. *'Controlling'* is essential for *re-scheduling* the remaining activities. However, an existing bar chart can be *modified* to depict the progress made. This can be done by showing the progress of each activity, by hatched lines along the corresponding bar of that activity. Generally, hatching is done in half the width of the bar.

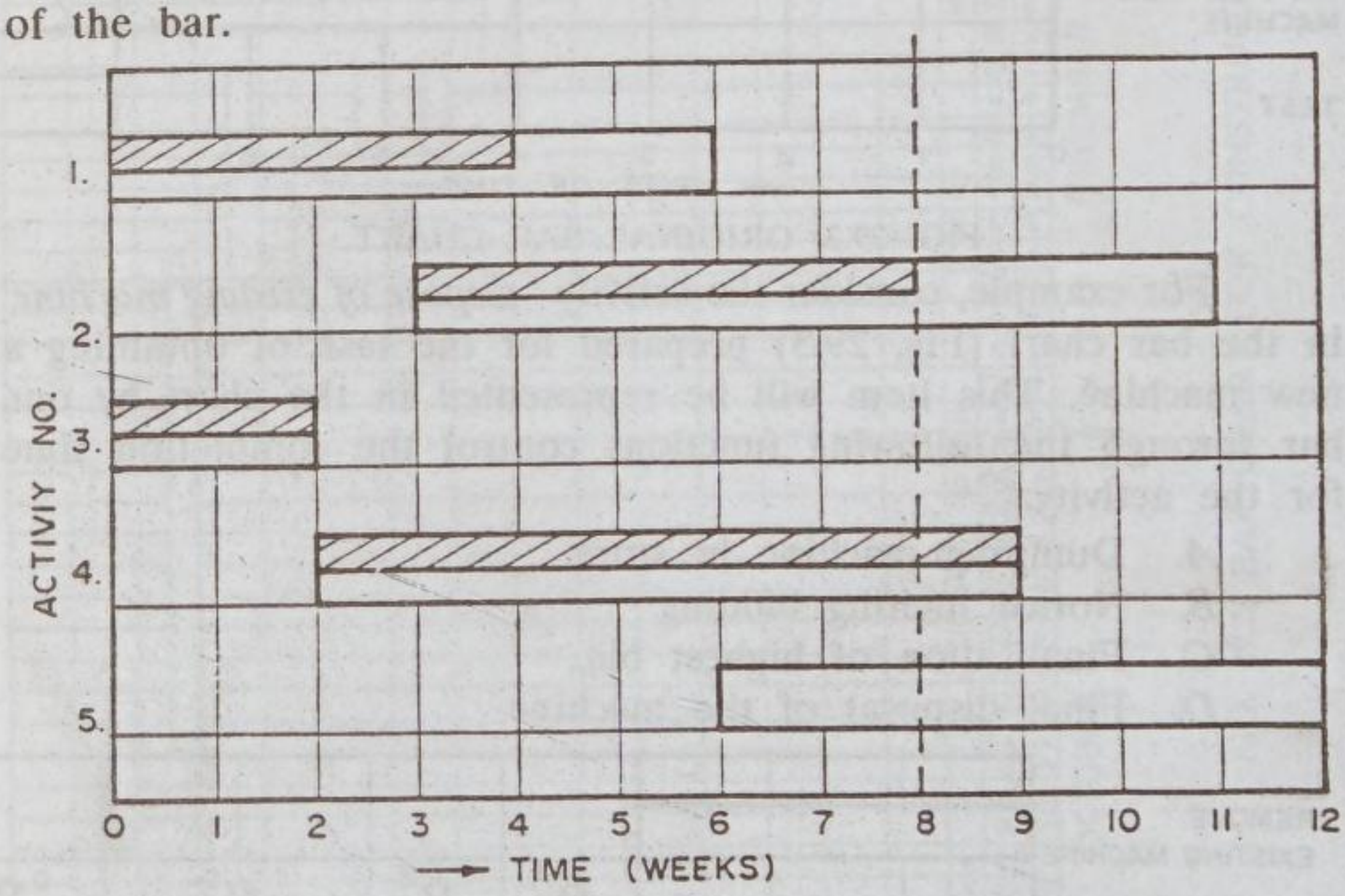


FIG. 29.5. PROGRESS OF ACTIVITIES ON BAR CHART.

For example, let us mark the progress made on the bar chart of Fig. 29.5, after 8 weeks of the start of the progress. Activity 1 had a total time allocation of 6 weeks. At the end of 8th week, only 4 weeks' work has been done ; that means that activity 1 is 4 weeks behind schedule. Activity 5 was wholly dependent on the completion of activity 1 ; the beginning of activity 5 will now be delayed by 4 weeks. Hence rescheduling of activity 5 is essential. Activities 2 and 3 are perfectly as per schedule. However, activity 4 is 1 week *ahead* of the schedule.

Sometimes, different colours are filled in the bars to show

various 'control informations', as indicated below :

Control information	*Colour*
Anticipated progress	black
Actual progress	green
Progress behind schedule	red

3. Activity inter-relationships

As indicated earlier, there are some activities of a project which are taken up concurrently, while there are others which can be taken up only after the completion of some other activity. The concurrent activities are represented by bars which run parallel to each other, or which overlap. The activities whose start and end depend on other activities are shown serially. In a project, there may be large number of activities which can start with a certain degree of concurrency. By merely depicting them by parallel lines, the inter-relationships between them cannot be *clearly* depicted. One cannot draw the conclusion that if two activities are scheduled for simultaneous or overlapping times, they are inter-dependent or completely independent. For example, take the project of laying a pipe line, consisting of following activities :

A. Excavating the trench	12 weeks
B. Laying and jointing the pipe	10 weeks
C. Refilling and compacting.	6 weeks

Activity *C* is dependent on *B* and *A*, while activity *B* is dependent on activity *A*. If all the activities are scheduled *serially*, it will take a very long time –28 weeks for completion. However, the activities can be staggered as shown in Fig. 29.6.

From bar chart of Fig. 29.6, we find that if activity *B* is started 4 weeks after activity *A*, activity *B* has 2 weeks work left after completion of activity *A*. Similarly, activity *C* has 2 weeks works left after completion of activity *B*. Now, if due to some circumstances, time of completion of activity *A* is delayed by 1 or 2 weeks, how will the activities *B* and *C* be affected ? This is not clearly portrayed by the bar chart, since inter-dependencies of the activities are not clearly indicated by bar charts.

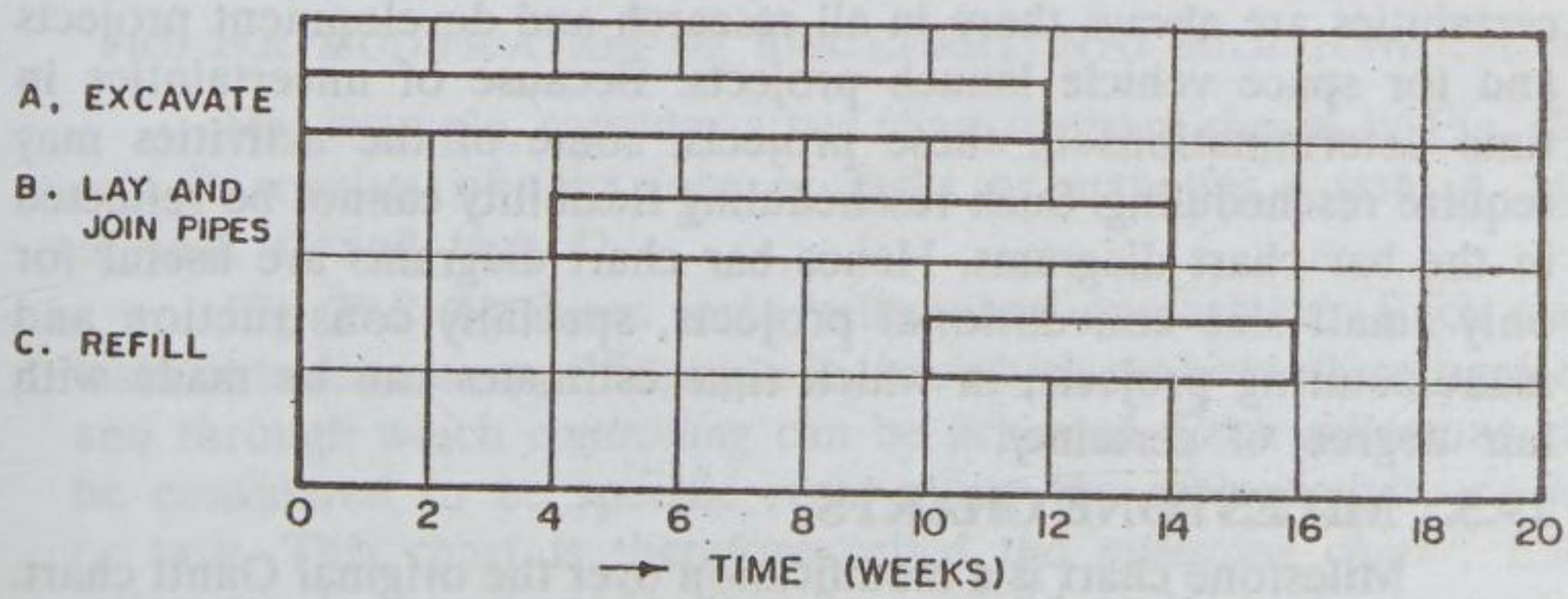


FIG. 29.6

This difficulty can be partly overcome by breaking each activity into a number of sections, so that the corresponding sections of various activity are *precisely* depicted interdependently. For example, let the jobs of Fig. 29.7 be divided into 4 *sections* . Since activity *B* is faster than *A,* and activity *C* is still faster, the shifting or staggering of these activities can be for more than 1 section. The modified bar chart is shown in Fig. 29.7 by depicting the completion of each section by 'mile stones' 1, 2, 3 and 4.

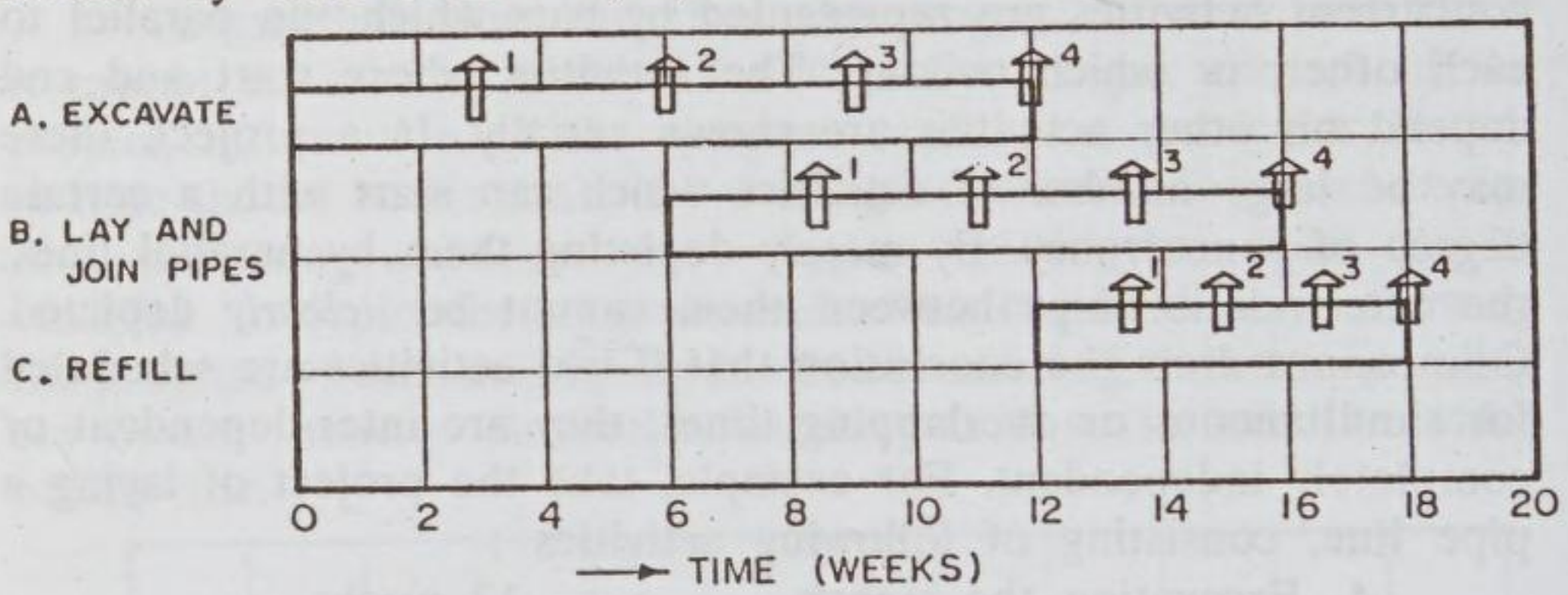


FIG. 29.7. MODIFIED BAR CHART.

For activity *A* (excavate), each section will require 3 weeks time. For activity *B*, each section will require $2\frac{1}{2}$ weeks time while for activity *C*, each section will require $1\frac{1}{2}$ weeks time of completion. If activity *B* is started 6 weeks after the start of activity *A*, it will mean that activity *B* will start after section 2 of activity *A* is already complete, and that activity *B* will require 4 weeks time after the completion of section 4 of activity *A*. Now the effects of delay in the work of any *section* of activity *A* on activity *B* can be easily found, and necessary control measures can be taken.

4. Time uncertainties. Bar charts are not all useful in those projects where there are uncertainties in determination or estimation of time required for the completion of various activities. Such uncertainties are always there in all research and development projects and for space vehicle launch projects. Because of uncertainties in time determinations in these projects, some of the activities may require rescheduling. Such rescheduling flexibility cannot be reflected in the bar chart diagrams. Hence bar chart diagrams are useful for only small size conventional projects, specially construction and manufacturing projects, in which time estimates can be made with fair degree of certainty.

29.5. MILESTONE CHARTS

Milestone chart is a modification over the original Gantt chart. *Milestones* are key events of a main activity represented by a bar;

these are specific points in time which mark the completion of certain portions of the main activity. These points are those which can be easily identified over the main bar. We have already seen that when a particular activity, represented by a bar on a bar chart is very long, the details lack. If however, the activity is broken or sub-divided into a number of sub-activities, each one of which can be easily recognised during the progress of the project, controlling can be easily done and inter-relationships between other similar activities can be easily established. The beginning and end of these sub-divided activities or tasks are termed as *milestones.*

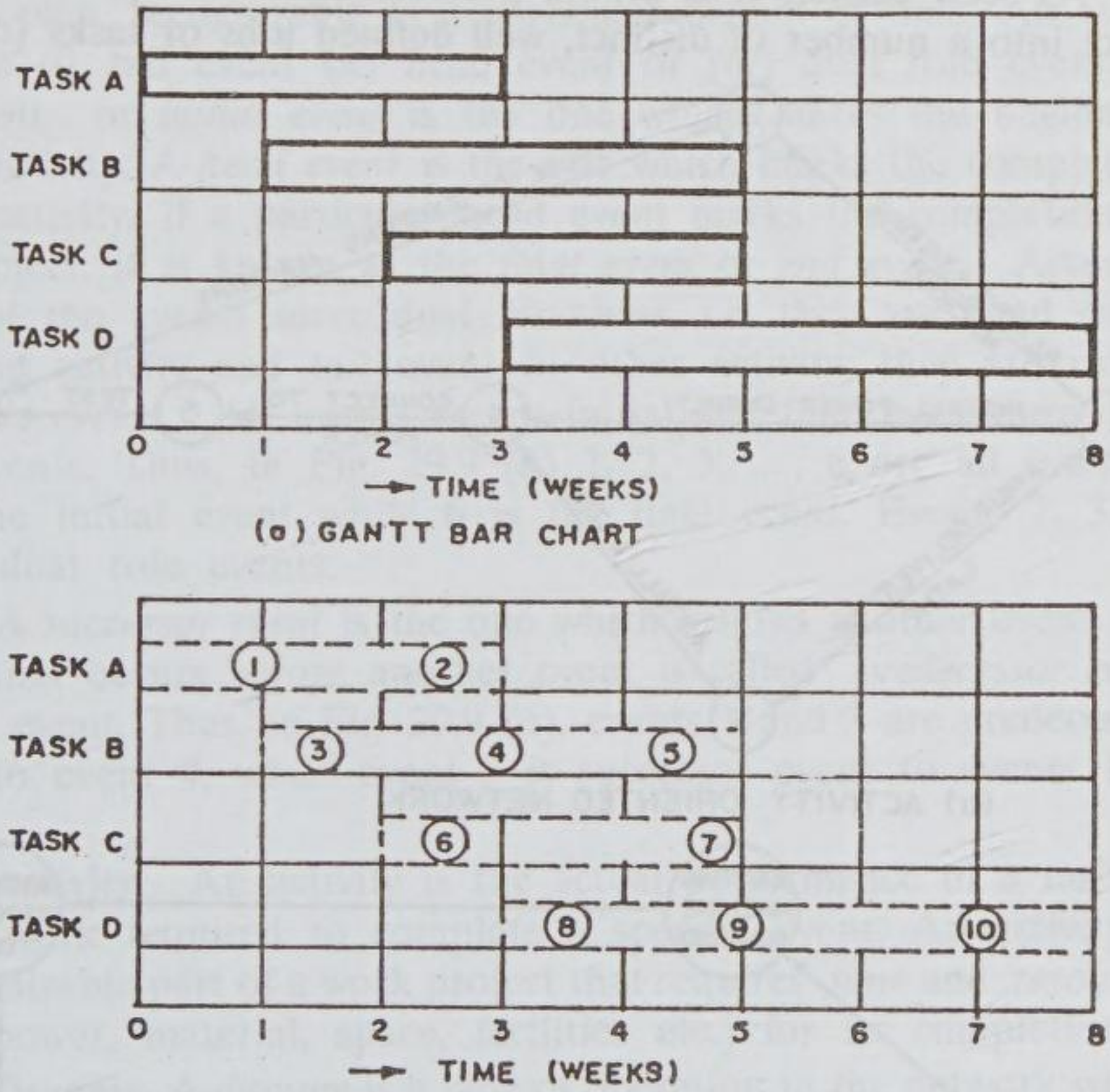


FIG. 29.8. MODIFICATION OF BAR CHART INTO MILESTONE CHART.

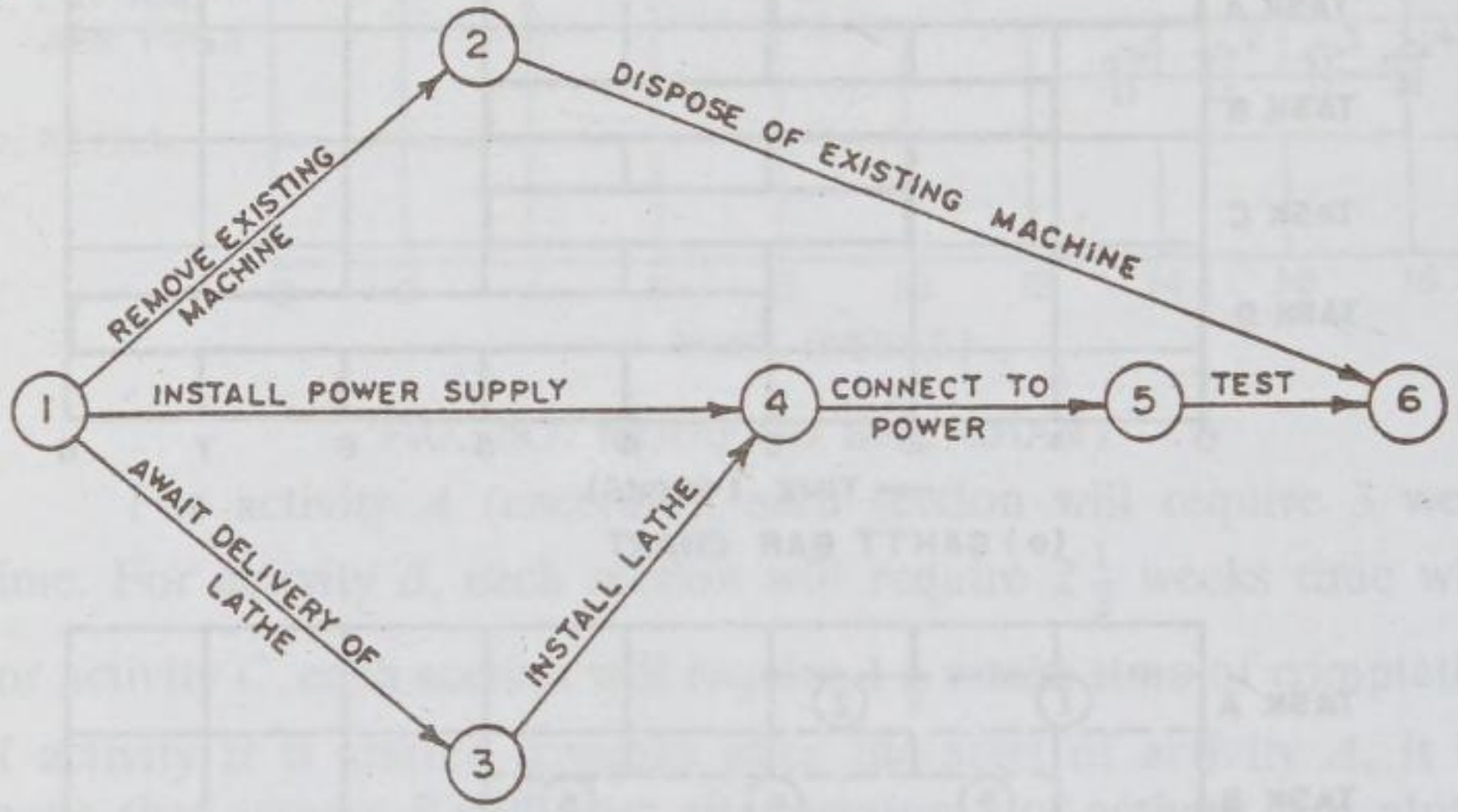
For example, consider a bar chart diagram shown in Fig. 29.8 (*a*). It consists of four jobs or tasks or activities : task *A*, task *B*, task *C* and task *D*.

Fig. 29.8 (*b*) shows some 'milestones' on each bar. Each main task contains some specific points in time which can be easily recognised, and through which *controlling* can be achieved. Each milestone can be considered to be specific *event* along the main activity or job or task. This chart is therefore called the *milestone chart* . Each milestone is represented either by a circle or by a square, and is serially marked.

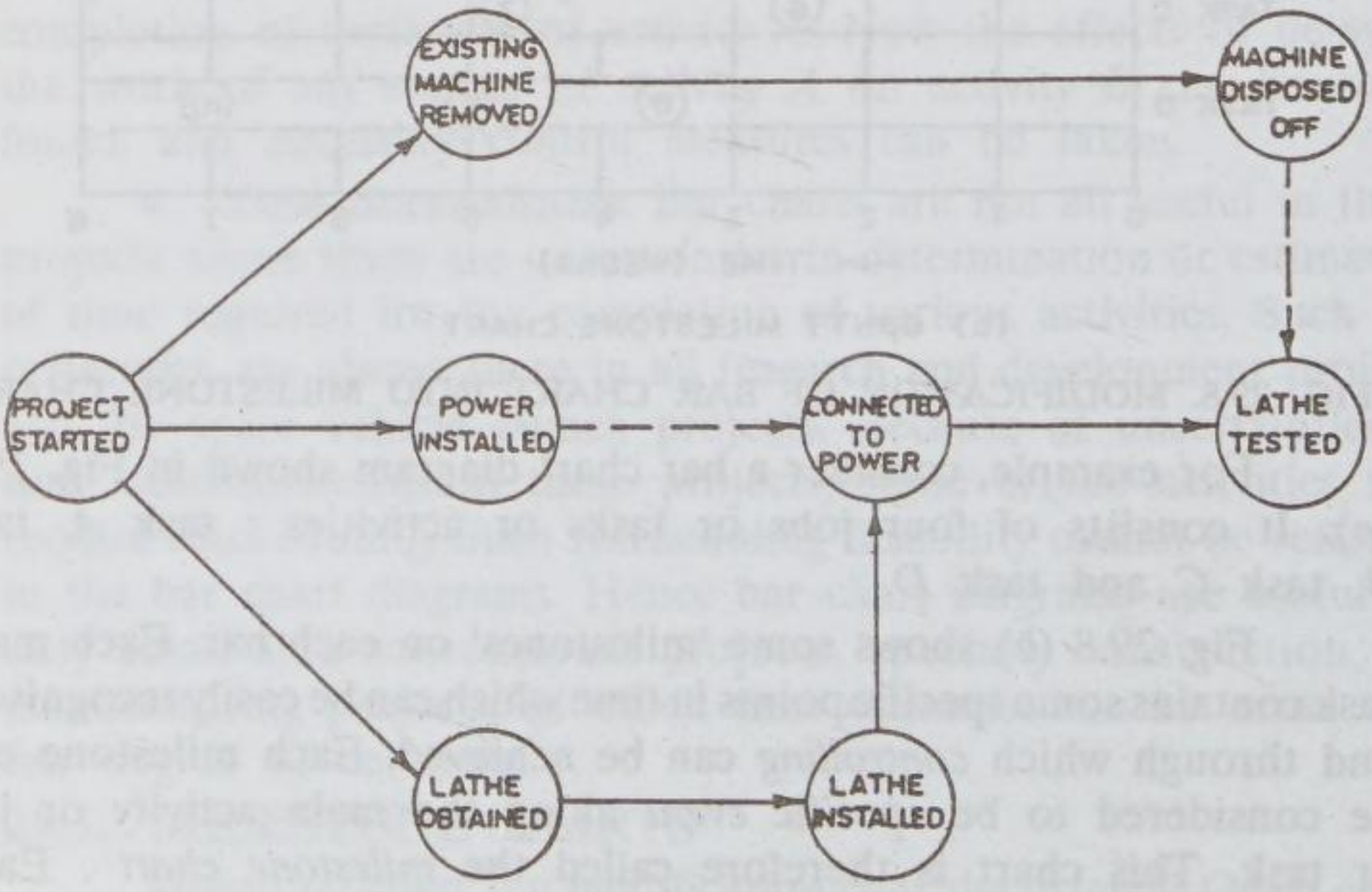
Though controlling can be better achieved with the help of milestone chart, it still possess the same deficiency contained by the bar chart–it does not show the inter-dependencies between the events. Within a task, the relationship between two specific milestones is revealed, but the relationship between and among milestones contained in different tasks is not indicated on the chart. For example, Gantt milestone chart [Fig. 29.8 (*b*)] does not indicate whether milestone 6 can be started before milestone 2 has been completed, or whether milestones of task *C* are at all dependent on milestones of task *A*.

29.6. ELEMENTS OF NETWORK

As seen earlier, it is always possible to break up the entire project into a number of distinct, well defined jobs or tasks (called



(a) ACTIVITY ORIENTED NETWORK



(b) EVENT ORIENTED NETWORK

FIG. 29.9. NETWORK DIAGRAMS.

activities). The beginning or end of each such activity constitutes an *event* of the project. A *network* is a *flow diagram* consisting of *activities* and *events*, connected logically and sequentially. In a network diagram, an activity is represented by arrows while events are represented, usually, by circles, as shown in Fig. 29.9.

A PERT network is *event oriented* , while CPM network is *activity oriented* .

Event. The commencement or completion of an activity is called an event. An event is that particular instant of time at which some specific part of a plan has been taken is to be achieved. More specifically, an event is a specific definable accomplishment in a project plan, recognizable at a particular instant of time. An event may be (*i*) tail event (*ii*) head event or (*iii*) dual role event. A *tail event* or *initial event* is the one which marks the beginning of an activity. A *head event* is the one which marks the completion of an activity. If a particular head event marks the completion of the project, it is known as the *final event* or *end event* . Actually, most of the events serve dual functions, *i.e.* they are head event to some activity and tail event to other activity, they are called *dual role events* . All events except initial and final events are dual role events. Thus, in Fig. 29.9 (*a*) 1, 2, 3, ..., 6 are all events ; 1 is the initial event while 6 is the final event. Events 2, 3, 4, 5 are dual role events.

A *successor event* is the one which follows another event. An event that occurs before another event is called *predecessor event* to that event. Thus, in Fig. 29.9 (*a*), events 1 and 3 are predecessor event to event 4, while event 5 is successor event to events 1, 3 and 4.

Activity. An activity is the actual performance of a task. It is the work required to complete a specific event. An activity is a recognizable part of a work project that requires *time* and *resources* (man power, material, space, facilities etc.) for its completion.

Dummy. A *dummy* is a type of operation in the network which neither requires any time nor any resources, but is merely a device to identify a *dependance* among operations. A dummy is thus a connecting link for control purposes or for maintaining uniqueness of activity.

Numbering the events : Fulkerson's rule

It is essential to number the events or node points. The activities joining the nodes can better be identified on the network by the event numbers or node numbers at the tail and head of the activity. The event numbering should be scientifically done so that they reflect their logical sequence. In a big network, the problem of numbering can be simplified if the rules devised by D.R. Fulkerson are followed.

The sequential numbering to the events may be assigned in the following steps :

1. There is a single initial event in a network diagram. This initial event will have arrows coming out of it and none entering it. Number this initial event as 1.

2. Neglect all the arrows emerging out of the initial event numbered 1. Doing so will apparenltly provide one or more *new* initial events.

3. Number these apparently produced new initial events as 2, 3, 4 etc.

4. Again neglect all emerging arrows from these numbered events ; this will create few more initial events.

5. Follow step 3.

6. Continue this operation until the last event, which has no emerging arrows, is numbered.

In bigger networks, where excessive modifications are frequently required to be made, *renumbering* can be avoided by numbering the events in the multiple of 10, *i.e.* numbering the events as 10 (initial), 20, 30 , 40 etc. If an event is added later, it can be assigned a number (such as 31, say) which lies between the number of immediate predecessor event and immediate successor event. This process of numbering is called *skip numbering*

Example 29.2. *Using Fulkerson rule, number the events of the network shown in Fig. 29.10.*

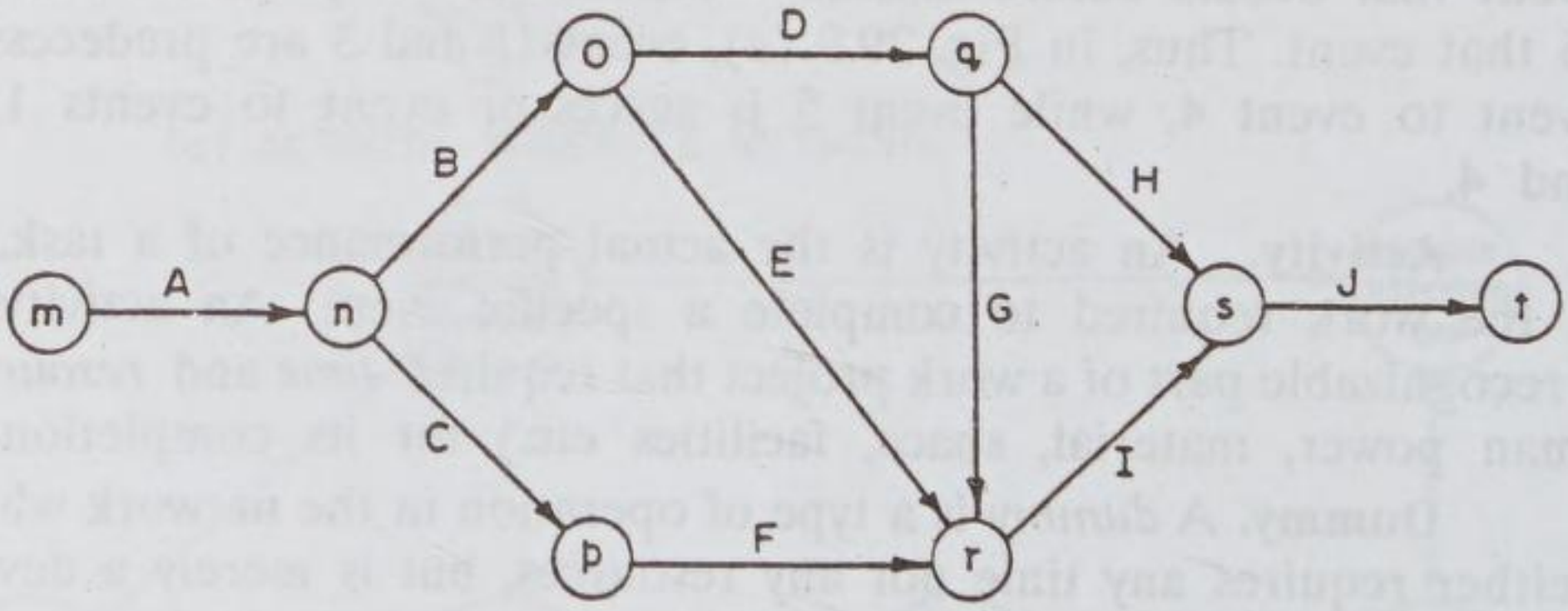


FIG. 29.10

Solution.

1. Event *m* is the initial event ; hence number it as 1.

2. Neglect the arrows (*A*) emerging out of the initial event Due to this event *n* will be the new initial event. Number it as 2

3. There are two arrows *B* and *C* emerging out of event No. 2. By neglecting these, two more new initial events are obtained at nodes *o* and *p* ; number these as 3 and 4 respectively.

4. Consider event 3. Neglect two arrows *D* and *E* emerging out of it. This will result in a new event at nodes *q* ; assign number

5 to it. Note that event *r* will still have arrows entering to it.

5. Consider event 5 and neglect arrows *G* and *H* coming out of it. This will result in two new initial events at nodes *r* and *s*. Since node *r* is predecessor event to node *s*, number it as 6. Number node *s* as 7.

6. Finally, event at node *t* has no arrow emerging out of it. Hence number this final event as 8.

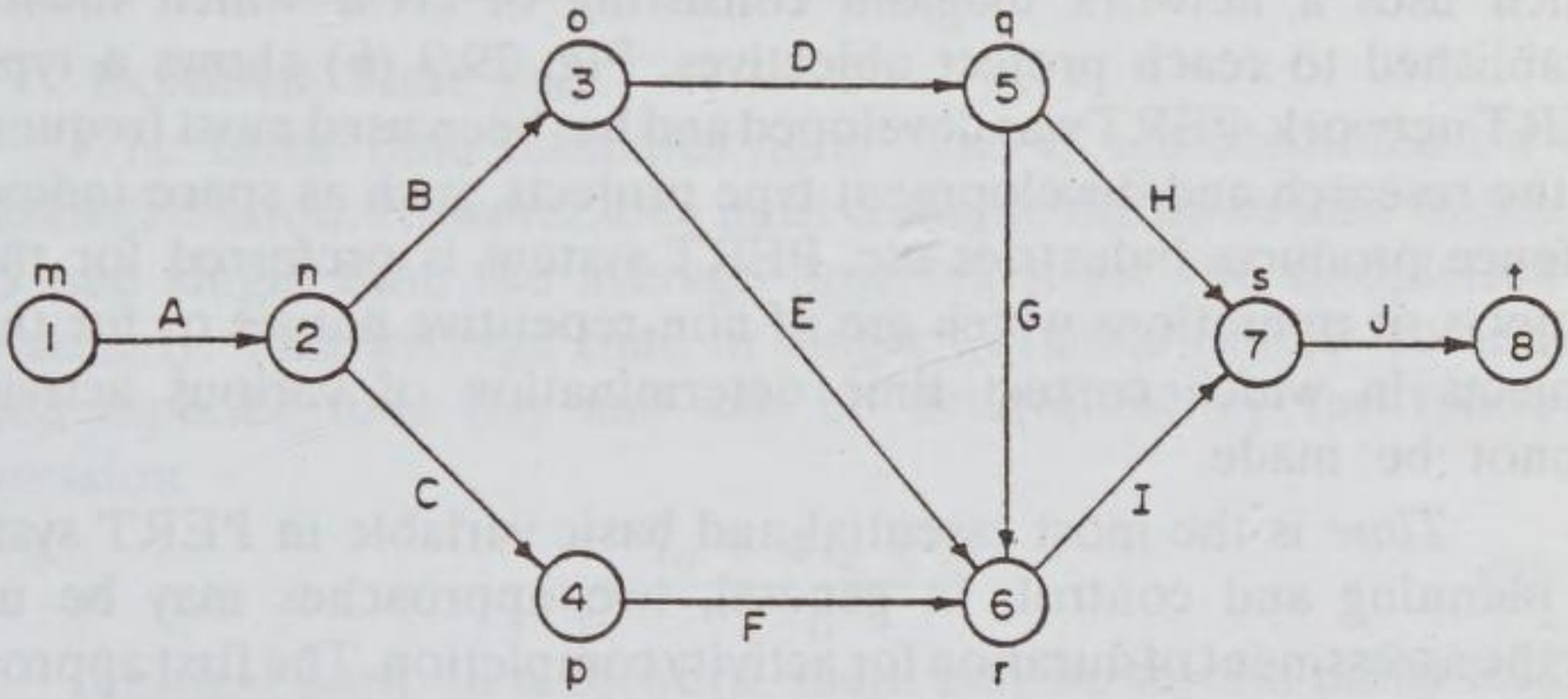


FIG. 29.11

The numbered network diagram is shown in Fig. 29.11.

Planning for network construction

Depending upon the sense of thinking with respect the end configuration of plan, networks can be constructed either by *forward planning* or by *backward planning* or by combination of both forward and backward planning.

1. **Forward planning.** In this method, the planner starts from the initial event and builds up the events and activities logically and sequentially until the end event is reached. In this method, while considering an activity, a planner asks himself the following questions :

What event comes next ?
What are dependent events ?
What events can take place concurrently ?

The answer to these questions is not that simple, specially in a complex situation.

2. **Backward planning.** In this method, the planner starts with the end event, and arranges the events and activities until the initial event is reached. Keeping the goal in view, the planner asks himself 'if we want to achieve this, what events or activities should have taken place ?'

3. **Combined planning.** In practice, a combination of both forward planning and backward planning is followed. At any stage, the planner may need transverse the network back and forth several times until it is found to be satisfactory. In this method, the planner

must ask himself the following questions, at any stage of network planning :

(*a*) What event or events must be completed before the particular event can start ?

(*b*) What event or events follows this ?

(*c*) What activities can be accomplished simultaneously ?

29.7. PERT NETWORKS

PERT stands for Programme Evaluation and Review Technique which uses a network diagram consisting of *event* which must be established to reach project objectives. Fig. 29.9 (*b*) shows a typical PERT network. PERT was developed and has been used most frequently in the research and development type projects, such as space industry, defence products industries etc. PERT system is preferred for those projects or operations which are of non-repetitive nature or for those projects in which correct time determination of various activities cannot be made.

Time is the most essential and basic variable in PERT system of planning and control. In general, two approaches may be used for the assessment of duration for activity completion. The first approach is the *deterministic approach* in which a single estimate of time gives reasonably accurate results, and this approach is followed by CPM users. The second approach is the probabilistic approach in which one may only be able to state limits within which it is virtually certain that the activity duration will lie. Between these limits, we must guess what is the probability of executing the activity. The second approach is followed by PERT planners.

Thus, to take the uncertainties into account, PERT planners make three kinds of time estimates :

(*i*) The optimistic time estimate.

(*ii*) The pessimistic time estimate

and (*iii*) The most likely time estimate.

1. The Optimistic Time Estimate (t_O)

This is the shortest possible time in which an activity can be completed, under ideal conditions. This particular time estimate represents the time in which we could complete the activity or job if everything went along perfectly, with no problems or adverse conditions. Better than normal conditions are assumed to prevail.

2. The Pessimistic Time Estimate (t_P)

It is the best guess of the maximum time that would be required to complete the activity. This particular time estimate represents the time it might take us to complete a particular activity if everything went wrong and abnormal situations prevailed. However, this estimate does not include possible effects of highly unusual catastrophies such as earthquakes, floods, fires etc.

3. The most Likely Time Estimate (t_L)

The most likely time or *most probable time* is the time that, in the mind of the estimater, represents the time the activity would most often require if normal conditions prevail. This time estimate lies between the optimistic and pessimistic time estimates. This time estimate reflects a situation where conditions are normal, things are as usual and there is nothing exciting.

Expected Time (t_E)

The three time estimates t_O, t_P and t_L are identified on the Beta-distribution. However, one must combine the three time estimates into one single time the average time taken for the completion of the activity. This average time or single workable time is commonly called *expected time* (t_E) and can be determined by the following expression

$$t_E = \frac{t_O + 4\,t_L + t_P}{6} \qquad ...(29.1)$$

Critical path. In a network, there may be several paths leading from starting events to the end event. The expected time along a path is the sum of expected times of individual activities along that path. Out of the several paths in a network, the one for which Σt_m is the largest, is known as the critical path

Example 29.3. *The network for a certain project is shown in Fig. 29.12. Determine the expected time for each of the path. Which path is critical ?*

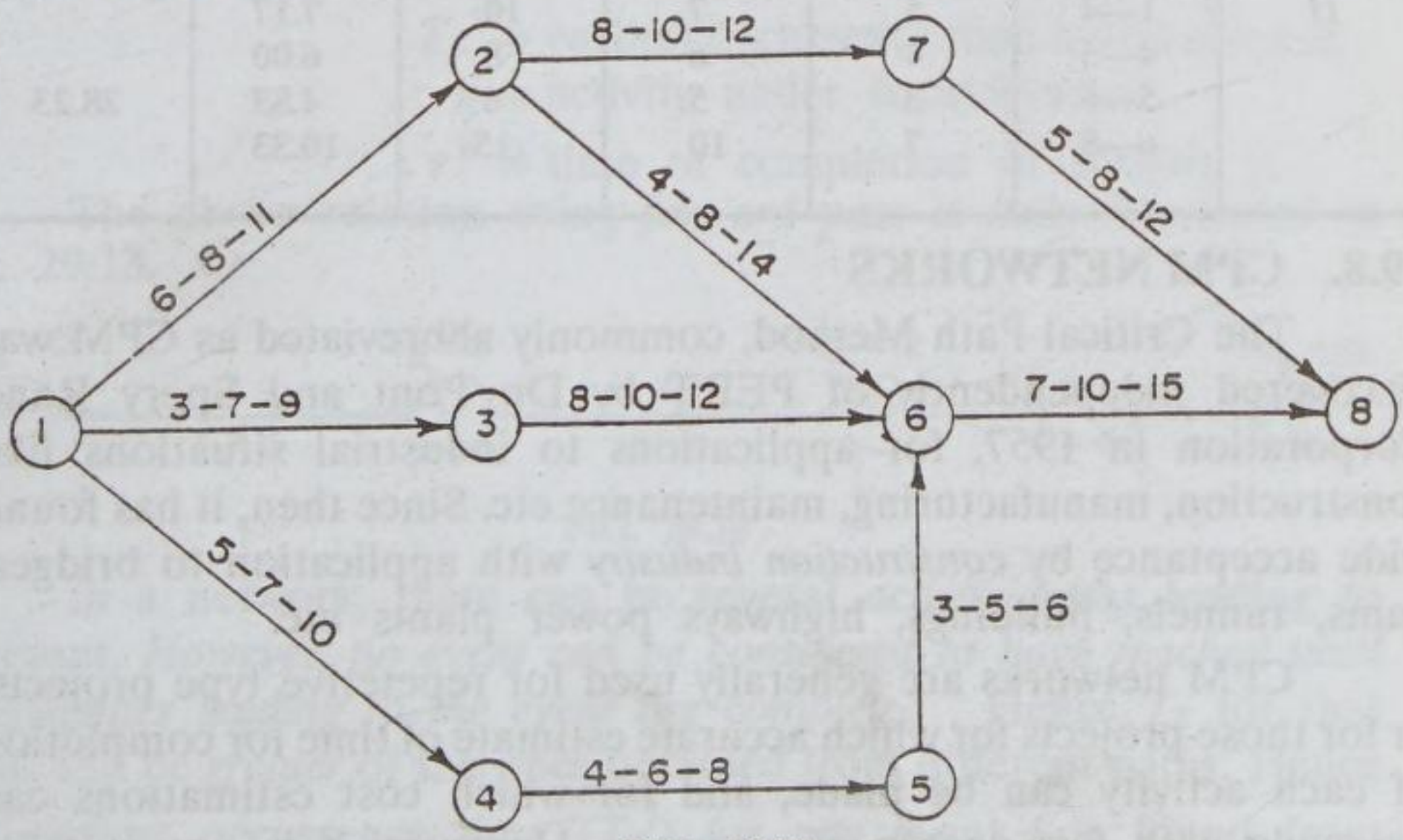


FIG. 29.12

Solution. In the network, event 1 is the starting event while event 8 is the end event. There are following four paths from the starting event to the end event :

Path A : 1–2–7–8
Path B : 1–2–6–8
Path C : 1–3–6–8
Path D : 1–4–5–6–8

In PERT analysis, the *expected time* t_E is taken as the basis for finding the critical path. The expected time t_E for each activity can be found from the equation

$$t_E = \frac{t_O + 4\,t_L + t_P}{6}$$

The expected time (t_E) for any path is equal to t_E of all activities. The computations are shown in Table 29.1, from which we find that path *D* is *critical* since $\Sigma\, t_E$ for this path is maximum.

TABLE 29.1

Path	*Activity*	t_O	t_L	t_P	t_E	Σt_E
A	1—2	6	8	11	8.17	
	2—7	8	10	12	10.00	26.34
	7—8	5	8	12	8.17	
B	1—2	6	8	11	8.17	
	2—6	4	8	14	8.33	26.83
	6—8	7	10	15	10.33	
C	1—3	3	7	9	6.67	
	3—6	8	10	12	10.00	27.00
	6—8	7	10	15	10.33	
D	1—4	5	7	10	7.17	
	4—5	4	6	8	6.00	
	5—6	3	5	6	4.83	28.23
	6—8	7	10	15	10.33	

29.8. CPM NETWORKS

The Critical Path Method, commonly abbreviated as CPM was discovered independently of PERT by Du Pont and Spery Rand Corporation in 1957, for applications to industrial situations like construction, manufacturing, maintenance etc. Since then, it has found wide acceptance by *construction industry* with application to bridges, dams, tunnels, buildings, highways power plants etc.

CPM networks are generally used for repetetive type projects, or for those projects for which accurate estimate of time for completion of each activity can be made, and for which cost estimations can be made with fair degree of accuracy. However, it is not suitable for research and development projects. As the name suggests, the *critical path* in the CPM method plays an important role in planning

and scheduling. A *critical path* is the time wise *longest path* in a network.

CPM networks are *activity oriented* while PERT networks are *event-oriented* . Essentially, this means that in activity oriented networks, the arrows representing activity or jobs are labelled with some description of activity. However, events constitute important control points in CPM networks. The events should therefore be so numbered that they reflect the logical sequence of the activities, and this can be best done by following Fulkerson' rule, discussed earlier.

Activity time estimate (t^{ij})

In CPM networks, the activity time estimates are based on deterministic approach in which we may assume that we know enough about each job or operation, so that a single time estimate of their duration is sufficiently accurate to give reasonable results. No uncertainties are taken into consideration. The time of completion of any activity *i-j* is denoted by symbol t^{ij}.

Earliest event time (T_E)

The *earliest event time* or *earliest occurrence time* is the earliest time at which an event can occur. It is the time by which all the activities discharging into the event under consideration are completed. For an activity path t_E for an event j can be computed from the relation

$$T_E^j = T_E^i + t^{ij} \qquad ...(29.2)$$

where T_E^i = earliest occurrence time for the tail event

T_E^j = earliest occurrence time for head event

ij = activity under consideration

t^{ij} = time of completion of activity ij.

The above relation using *forward pass* is fully illustrated in Fig. 29.13.

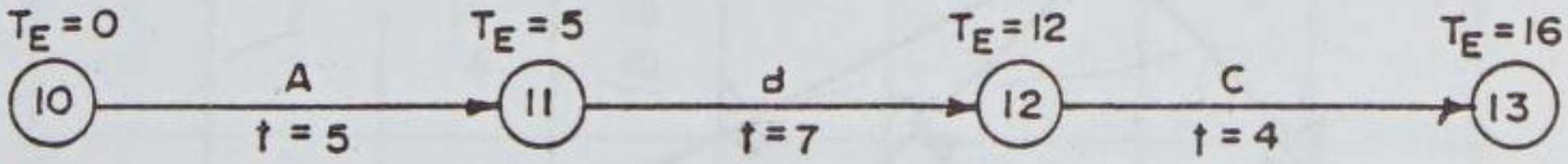


FIG. 29.13

In a network, there can be several *activity paths* leading to an event. *However, no event can be considered to have reached until all activities leading to the event are completed* . Hence T_E for that event will be *greater* of the ones obtained from different paths. Hence the earliest occurrence time (T_E^j) for any event j is found from the expression

$$T_E^j = (T_E^i + t^{ij})_{max} \qquad ...(29.3)$$

Latest allowable occurrence time (T_L)

The *latest allowable time* (T_L) or the *latest event time* is the latest time by which an event must occur to keep the project on schedule. If the *scheduled completion time* (T_s) of the project is given, the latest *event time* of the end event will be equal to T_s. If T_s is not specified, then T_L of end event can be taken equal to its T_E.

The *latest event time* for an activity is computed by starting from head event and using the *backward pass* . In general, for an *activity path*, T_L^j for a predecessor event i can be determined from the known value of T_L^j of a successor event, by the relation

$$T_L{}^i = T_L{}^i - t^{ij} \qquad ...(29.4)$$

For a network, where several activity paths originate or radiate from the event under consideration ; T_L for the event will be different from different path. However, minimum value of T_L will be the latest event time for that event. Thus,

$$T_L{}^i = (T_L{}^i - t^{ij})_{min.} \qquad ...(29.5)$$

Combined tabular computations for T_E and T_L

As stated earlier, the *earliest event time* (T_E) is computed using *forward pass* (Eqs. 29.2 and 29.3) while the *latest event time* (T_L) is computed using *backward pass* (Eqs 29.4 and 29.5). A combined tabular form, for the network shown in Fig. 29.14 is illustrated in Table 29.2.

Column 1 of Table 29.2 gives the event number, starting with the initial event and proceeding in the direction of increasing numbers of the events. Column 2 gives the predecessor events while column 6 gives the successor events to the events of column 1. These columns

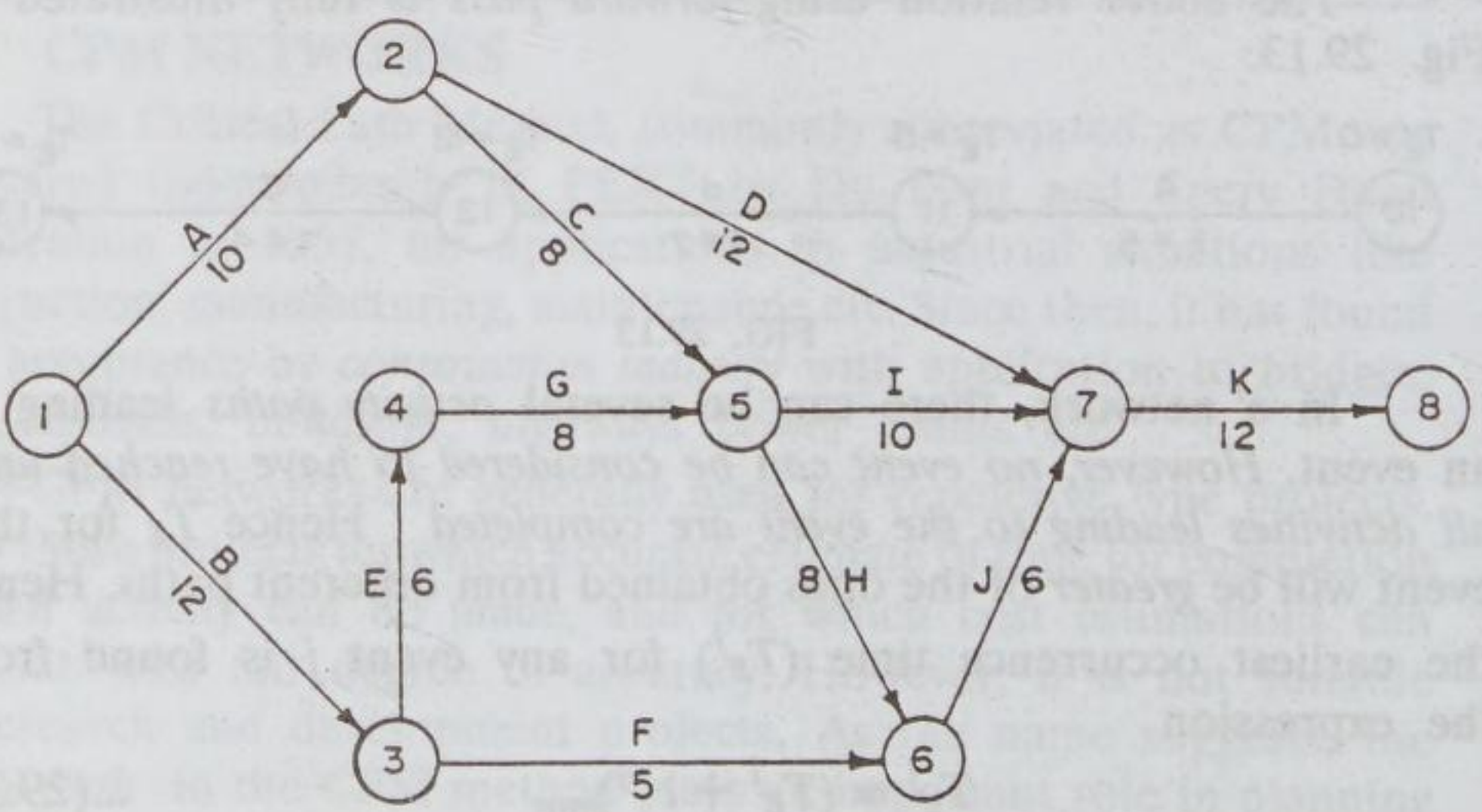


FIG. 29.1'

are completed first, using the network. An event under consideration (column 1) may have one or more than one predecessor events (column 2), and one or more than one successor events (column 6). A horizontal line is drawn after entering all the predecessor events and successor events to every event of column 1.

TABLE 29.2 COMPUTATIONS OF T_E AND T_L

Event No.	*Earliest event time* (↓)				*Latest event time* (↑)			
	Predecessor event (i)	t^{ij}	TE^i	T_E	*Successor Event (j)*	t^{ij}	$T_L{}^i$	T_L
					(i)			
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*	*(7)*	*(8)*	*(9)*
1	—	—	0	0	2 3	10 12	8 0	0
2	1	10	10	10	5 7	8 12	18 28	18
3	1	12	12	12	4 6	6 5	12 29	12
4	3	6	18	18	5	8	18	18
5	2 4	8 8	18 26	26	6 7	8 10	26 30	26
6	3 5	5 8	17 34	34	7	6	34	34
7	2 5 6	12 19 6	22 36 40	30	8	12	40	40
8	7	12	52	52	—	—	52	52

Then, computations are done for the *earliest event time* (T_E) in columns 3, 4 and 5. Column 3 is for the activity time t^{ij} where j is the event under consideration (column 1) and i is the predecessor event (column 2). $T_E{}^j$ is computed from the relation

$$T_E^j = T_E{}^i + t^{ij}$$

Where there are more than one predecessor events, several value of $T_E{}^j$ are obtained, which are entered in column 4. The maximum

value of T_E^j is underlined. This underlined value is the *appropriate* value of the earliest event time for the event under consideration (column 1), and is entered as T_E in column 5. For the computation of T_E, we thus use *forward pass* starting from initial event and proceeding in the downward direction (↓) in the Table.

Then we compute the *latest event* (*occurrence*) *time* of the same events under consideration (column 1), in columns 7, 8 and 9. Column 7 is the activity time t^{ij} where i is the event under consideration and j is the successor event (column 6). T_L^i is computed from the relation

$$T_L^i = T_L^j - t^{ij}$$

Computations are done by *backward pass*, starting with the *end event* and proceeding *upwards* (↑) in the Table. If T_S is not given, T_L of the last event is taken equal to its T_E. Where there are more than one successor events, several values of T_L^i are obtained, which are entered in column 8. The minimum value of T_L^i is underlined. This underlined value is the appropriate value of the *latest event time* for the event under consideration, and is entered as T_L in column 8.

Thus, for each of the activities of column 1, T_L is given in column 5 while T_L is given in column 9. These values are also marked on Fig. 29.15.

Start and finish times of activities

CPM networks are activity oriented. The following activity times are useful for network computations : (*i*) Earliest Start Time (*ii*) Earliest Finish Time (*iii*) Latest Start Time and (*iv*) Latest Finish Time.

(*i*) *Earliest Start Time* (*EST*). Earliest start time of an activity is the earliest time by which it can commence. This is naturally equal to the earliest event time associated with the tail of the activity arrow. Thus, for an activity *i-j*.

$$EST = T_E^i \qquad ...(29.6)$$

(*ii*) *Earliest Finish Time* (*EFT*) If an activity proceeds from its early start time and takes the estimated duration for completion, then it will have an early finish. Hence *EFT* for an activity is defined as the earliest time by which it can be finished. This is evidently equal to the earliest start time plus estimated duration of the activity:

$$\text{EFT} = \text{earliest start time} + \text{activity duration}$$

or

$$\text{EFT} = T_E^i + t^{ij} \qquad ...(29.7)$$

(*iii*) *Latest Start Time* (*LST*) Latest start time for an activity is the *latest time* by which an activity can be started without delaying

the completion of the project. For 'no delay' condition to be fulfilled it should be naturally equal to the latest finish time (LFT) minus the activity duration.

$$\therefore \quad \text{LST} = \text{LFT} - \text{Activity duration}$$

Since the Latest Finish Time LFT is equal to T_L^j , (See Eq. 29.8), we have

$$\text{LST} = T_L^j - t^{ij} \qquad ...(29.8)$$

(*iv*) *Latest Finish Time* (LFT) The latest finish time for an activity is the latest time by which an activity can be finished without delaying the completion of the project. Naturally, the latest finish time for an activity will be equal to the latest allowable occurrence time for the event at the head of the arrow. Hence

$$\text{LFT} = \text{Latest event time at the head of activity arrow}$$

or $$\text{LFT} = T_L^j \qquad ...(29.9)$$

Floats*

Float denotes the range within which an activity start time or its finish time may fluctuate without affecting completion of the project. Floats are of the following types : (*i*) Total float (*ii*) Free float (*iii*) Independent float and (*iv*) Interfering float.

(*i*) **Total Float** (F_T) In certain activities, it will be found that there is a difference between maximum time available and the actual time required to perform the activity. This difference is known as the *total float* . In other words, it is the excess of the *maximum available* time over the activity time. Thus

$$F_T = (T_L^j - T_E^i) - t^{ij} \qquad ...(29.10)$$

or $$F_T = \text{LST} - \text{EST} = \text{LFT} - \text{EFT} \qquad ...(29.10\ a)$$

(*ii*) **Free float** (F_F) Free float is that portion of positive total float that can be used by an activity without delaying any succeeding activity (or without affecting the total float of the succeeding activity). In other words, it is the *excess of available time* over the *activity time when all jobs start as early as possible.*

Thus

$$F_F = (T_E^j - T_E^i) - t^{ij} \qquad ...(29.11)$$

or $$F_F = T_E^j - \text{EFT} \qquad ..(29.11a)$$

(*iii*) **Independent Float** (F_{ID}) Independent float gives us an idea about the excess time that exists if the preceding activity ends

---

*Note. For detailed physical concept of the floats readers may refer the book 'Project Planning and Control with PERT and CPM' by Dr. B.C. Punmia and K.K. Khandelwal, Published by Laxmi Publications, Delhi--6.

late as possible and the succeeding activity starts as early as possible. The independent float of an activity is thus the *excess* of *minimum available time* over the *activity time* .

Thus, $$F_{ID} = (T_E^j - T_L^i) - t^{ij} \quad ...(29.12)$$

or $$F_{ID} = F_F - S_i \quad ...(29.12a)$$

where S_i= tail event slack= $T_L^i - T_E^j$. ...(29.13)

Example 29.4. *For the network shown in Fig. 29.14, compute the various activity times and also compute* (*i*) *total float* (*ii*) *free float and* (*iii*) *independent float for each activity.*

Solution. The value of T_E and T_L for each event has already been computed in Table 29.2, and these are shown marked in Fig. 29.15. Columns 1 and 2 of Table 29.3 give the activities $(i - j)$ and their durations (t^{ij}) respectively. Columns 3, 4, 5 and 6 give the values of EST, EFT, LST and LFT which can be computed from Eqs 29.6, 29.7, 29.8 and 29.9 respectively.

Column 7 and 8 give F_T and F_F which can be computed from Eqs. 29.10 (*a*) and 29.11 (*a*) respectively. Column 9 gives F_{ID} which is computed from Eqs 29.12 (*a*), in which F_F is taken from Column 8, while S_i is computed directly from Fig. 29.15. For example, for activity 2–5, tail event slack=$T_L - T_E$=18−10=8, while $F_F = 8$, hence F_{ID}=8−8=0. However , for activity 2–7, $F_F = 18$ while S_i=18−10=8. Hence $F_{ID} = 18 - 8 = 10$.

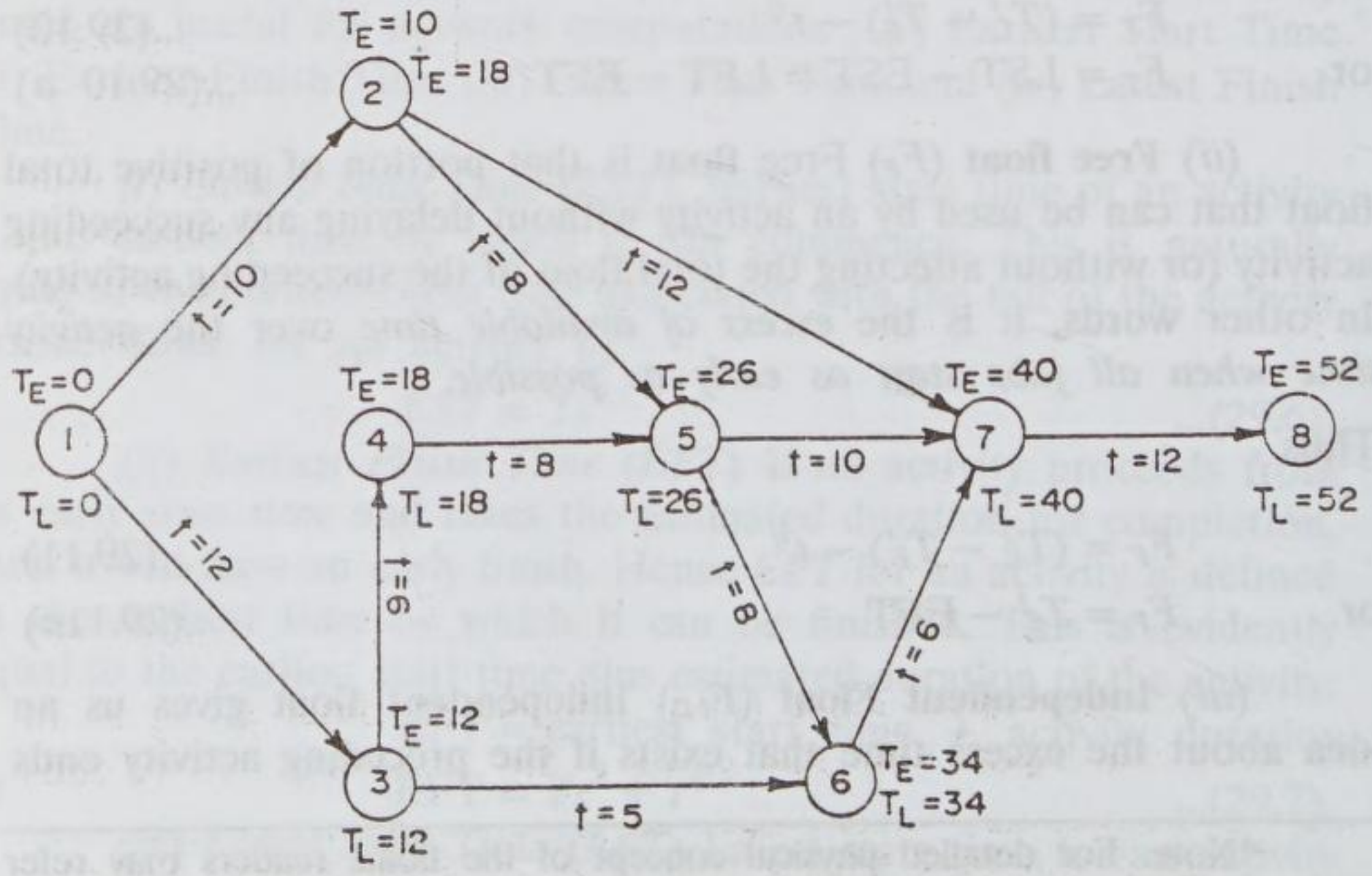


FIG. 29.15

TABLE 29.3

Activity (i-j)	*Dura-tion* t^{ij}	*Earliest*		*Latcst*		*Total float* F_T	*Free float* F_F	*Indepen-dent float* F_{ID}
		Start time (EST)	*Finish time (EFT)*	*Start time (LST)*	*Finish time (LFT)*			
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*	*(7)*	*(8)*	*(9)*
1–2	10	0	10	8	18	8	0	0
1–3	12	0	12	0	12	0	0	0
2–5	8	10	18	18	26	8	8	0
2–7	12	10	22	28	40	18	18	10
3–4	6	12	18	12	18	0	0	0
3–6	5	12	17	29	34	17	17	17
4–5	8	18	26	18	26	0	0	0
5–6	8	26	34	26	34	0	0	0
5–7	10	26	36	30	40	4	4	4
6–7	6	34	40	34	40	0	0	0
7–8	12	40	52	40	52	0	0	0

29.9. CRITICAL ACTIVITIES AND CRITICAL PATH

An activity is called *critical* when its *total float* is zero, *i.e.*, when time availability is equal to the activity duration. The *critical* path, as already defined, is the longest path through the network and time along this path gives the project duration : Critical path

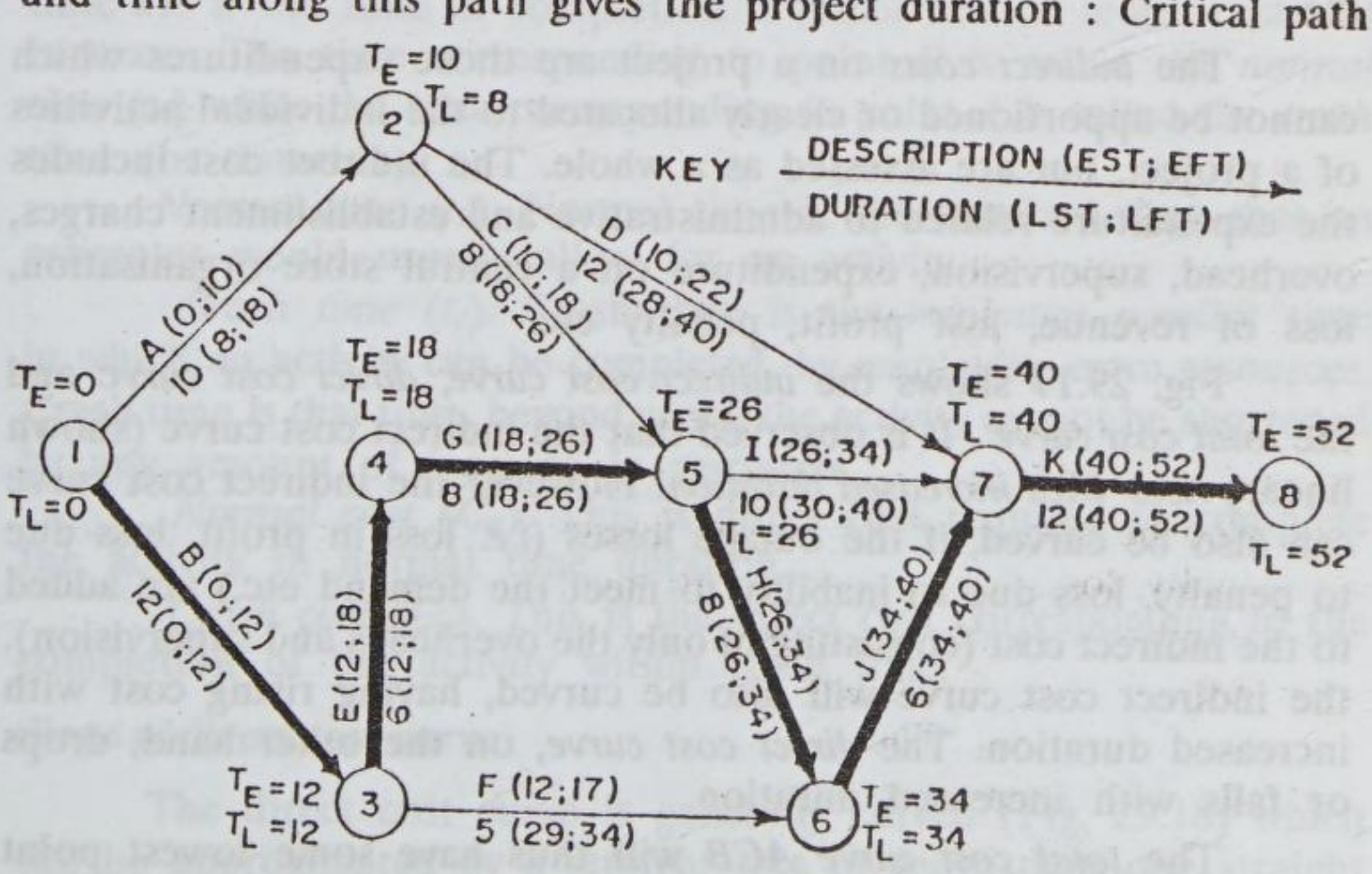


FIG. 29.16. CRITICAL PATH.

joins those activities which are *critical.* Critical path can be easily determined with the help of *total float calculations.* The activities on the critical path are those activities that have total float equal to zero. The activities that control the project duration are the ones that have zero total float and form continuous chain (or path) starting at the first node and ending with the last node.

The critical path for the network of Fig. 29.14 and 29.15 is shown by thick lines in Fig. 29.16, along with the activities start and finish times marked. It is to be noted that there can be more than one critical path in a network and the *degree of criticality* can be assigned to a particular path.

29.10. CPM : COST MODEL

CPM makes use of the cost estimate along with time estimate and provides a schedule for completing the activities at the minimum total cost. The object of the network analysis is *also* to assess the possibility of arriving at a feasible and desirable *time-cost relationship.* The over all project duration can be reduced by reducing the duration of only the critical activities in the project network.

Project cost

Total project cost is the sum of the following :

(*i*) *The direct cost*

(*ii*) *The indirect cost*

The *direct costs* are those expenditures which are directly chargeable to and can be identified specially with the activities of the project. These include labour cost, material cost, equipment cost etc.

The *indirect costs* on a project are those expenditures which cannot be apportioned or clearly allocated to the individual activities of a project, but are assessed as a whole. The indirect cost includes the expenditure related to administrative and establishment charges, overhead, supervision, expenditure on a central store organisation, loss of revenue, lost profit, penalty etc.

Fig. 29.17 shows the *indirect cost curve, direct cost curve* and the *total cost curve* . It is observed that the indirect cost curve (shown linear) *rises* with *increased duration.* However the indirect cost curve can also be curved, if the outage losses (*i.e.* loss in profit, loss due to penalty, loss due to inability to meet the demand etc.) are added to the indirect cost (consisting of only the overheads and supervision), the indirect cost curve will also be curved, having rising cost with increased duration. The *direct cost curve*, on the other hand, drops or falls with increased duration.

The *total cost curve ACB* will thus have some lowest point *C*, indicating the *minimum cost* and the corresponding time (t_o) gives

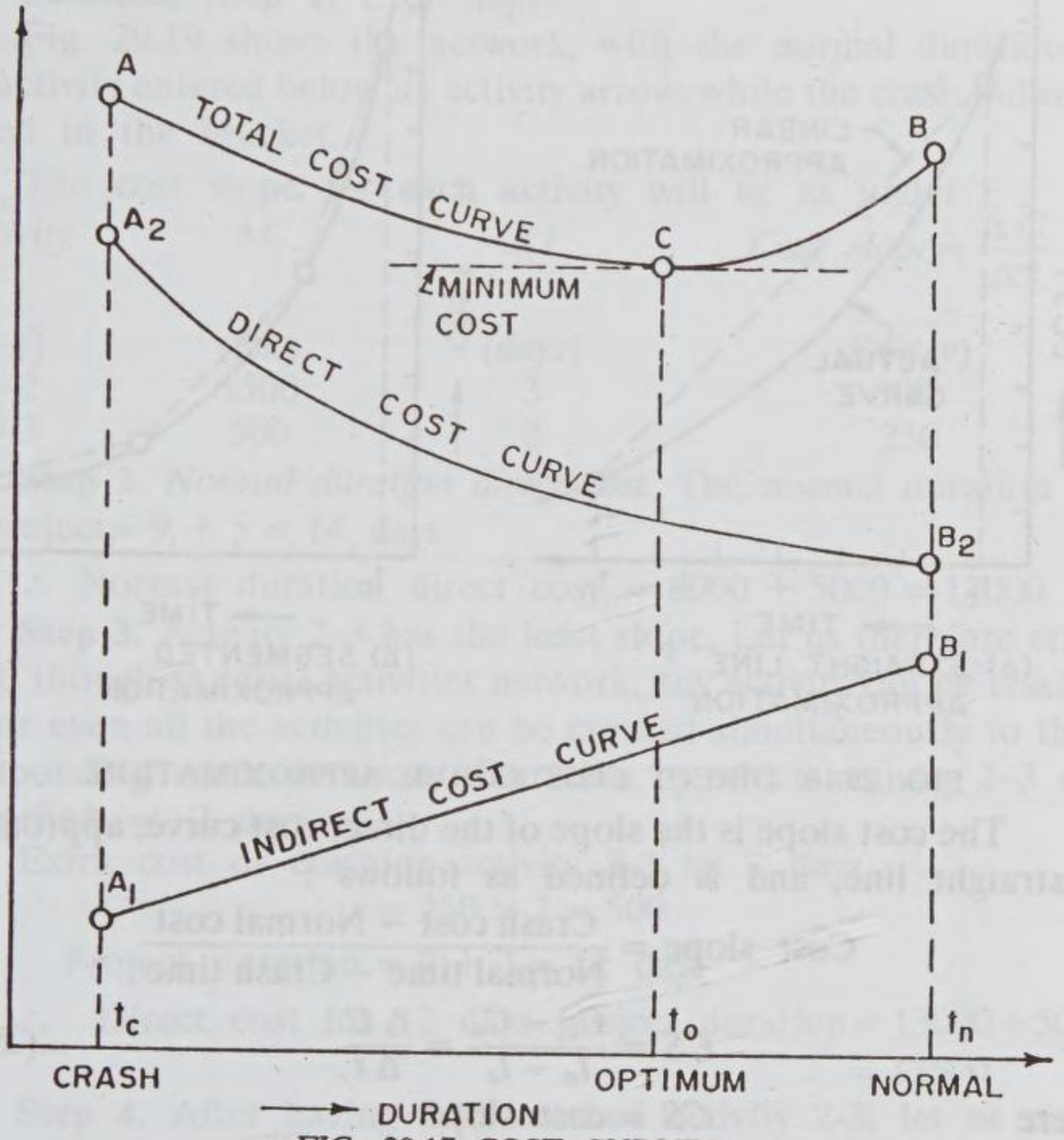


FIG. 29.17 COST CURVES

the *optimum time*. If the project is not completed within the optimum time *i.e.* if the time of completion is either less or more, cost will increase. The time corresponding to point *B* is called the *normal time* (t_n) while the time corresponding to point *A* is called the *crash* time (or duration), t_c.

Normal time (t_n). Normal time is the standard time that an estimater would usually allow for an activity.

Crash time (t_c). Crash time is the *minimum possible* time in which an activity can be completed, by employing extra resources. Crash time is that time, beyond which the activity cannot be shortened by any amount of increase in resources.

Normal cost (C_n). This is direct cost required to complete the activity in normal time duration.

Crash cost (C_c). This is the direct cost corresponding to the completion of the activity within crash time.

Slope of direct cost curve

The direct cost curve is generally curved (Fig. 29.18) which can be approximated by a straight line or more than one straight line, depending upon the flatness of the curve.

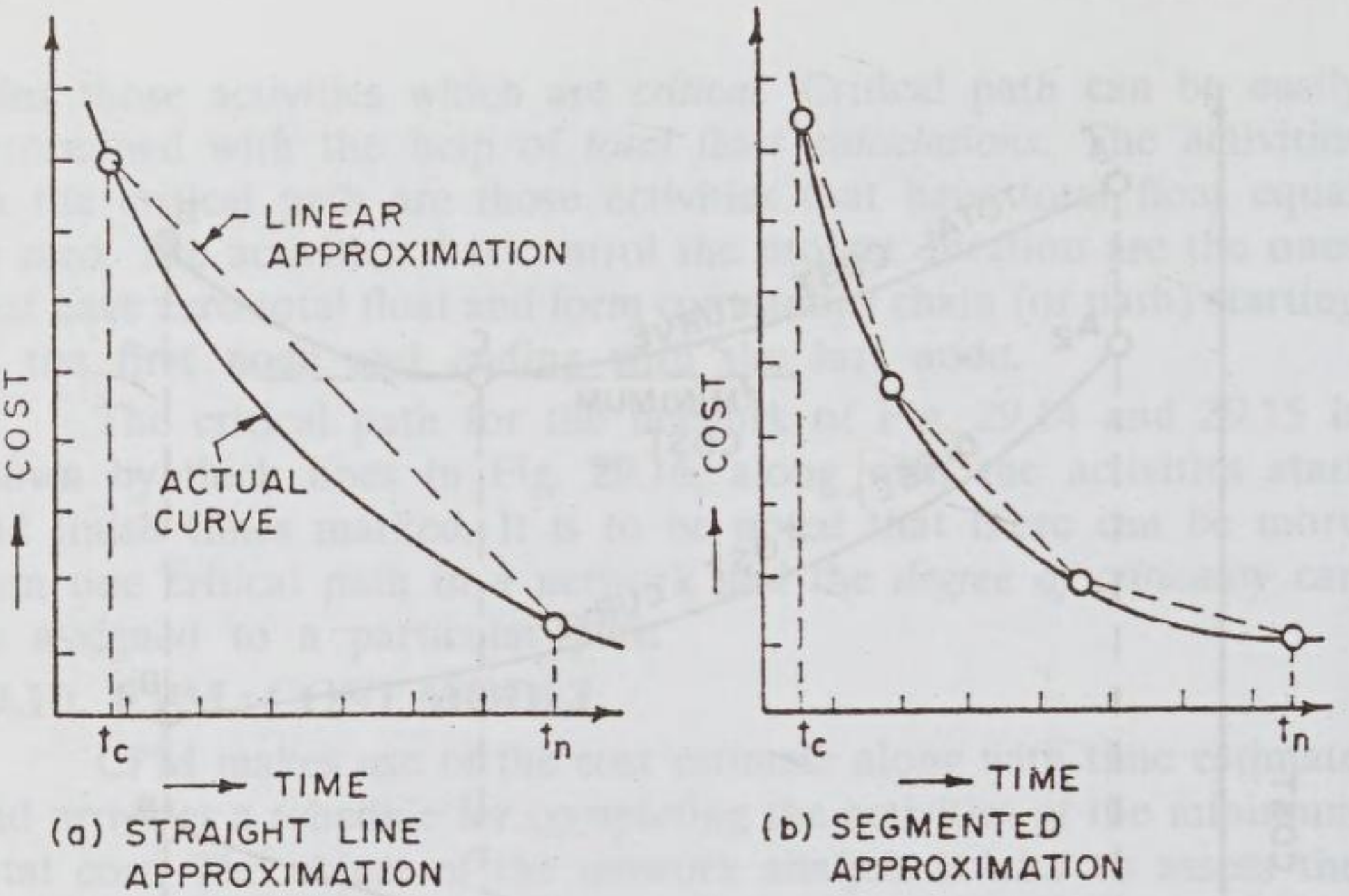


FIG. 29.18. DIRECT COST CURVE APPROXIMATION.

The cost slope is the slope of the direct cost curve, approximate as straight line, and is defined as follows :

$$\text{Cost slope} = \frac{\text{Crash cost} - \text{Normal cost}}{\text{Normal time} - \text{Crash time}}$$

or
$$CS = \frac{C_c - C_n}{t_n - t_c} = \frac{\Delta C}{\Delta t} \qquad ...(29.14)$$

where CS = cost slope

ΔC = increase in the cost

Δt = decrease in time duration.

The method of cost optimisation is illustrated in Example 29.5*.

Example 29.5. *Table 29.4 gives the information about various activities of network shown in Fig. 29.19.*

TABLE 29.4

Activity	*Normal duration (days)*	*Normal cost (Rs.)*	*Crash duration (days)*	*Crash cost (Rs.)*
1—2	9	8000	6	9500
2—3	5	5000	3	5500

The project overhead costs are @ Rs. 300.0 per day. Determine (a) direct cost - duration relationship. (b) total cost -duration relationship and the corresponding least cost plan (network).

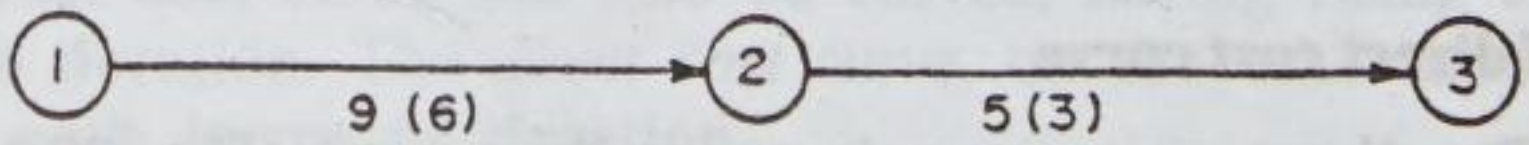


FIG. 29.19

*For detailed study, the reader may refer the book 'Project Planning and Control with PERT and CPM' by Dr. B.C. Punmia and K.K. khandelwal.

Solution, Step 1. *Cost slopes*

Fig. 29.19 shows the network, with the normal duration of each activity entered below its activity arrow, while the crash duration entered in the bracket.

The cost slope for each activity will be as under :

Activity	ΔC	Δt	$Cost\ slope = \frac{\Delta C}{\Delta t}$
(i–j)	*(Rs.)*	*(days)*	*(Rs/day)*
1–2	1500	3	500
2–3	500	2	250

Step 2. *Normal duration direct cost.* The normal duration for the project $= 9 + 5 = 14$ days.

$\therefore$ Normal duration direct cost $= 8000 + 5000 = 13000$.

Step 3. Activity 2–3 has the least slope. Let us therefore crash it first, though in serial-activities network, any activity can be crashed first, or even all the activities can be crashed simultaneously to their corresponding crash durations. Duration by which activity 2–3 can be crashed = 2 days.

Extra cost of crashing activity 2-3 by 2 days

$$= 250 \times 2 = 500$$

Project duration $= 9 + 3 = 12$ days

$\therefore$ Direct cost for 12 days project duration $= 13000 + 500$

$$= 13500$$

Step 4. After having fully crashed activity 2-3, let us crash activity 1-2 from its normal duration 9 days to its crash-duration of 6 days.

$$\Delta t = 9 - 6 = 3 \text{ days.}$$

Extra cost of crashing $= 3 \times 500 = 1500$

Project duration $= 6 + 3 = 9$ days.

$\therefore$ Direct cost for 9 days project duration $= 13500 + 1500$

$$= 15000$$

The corresponding net work with all the activities crashed as shown in Fig 29.20.

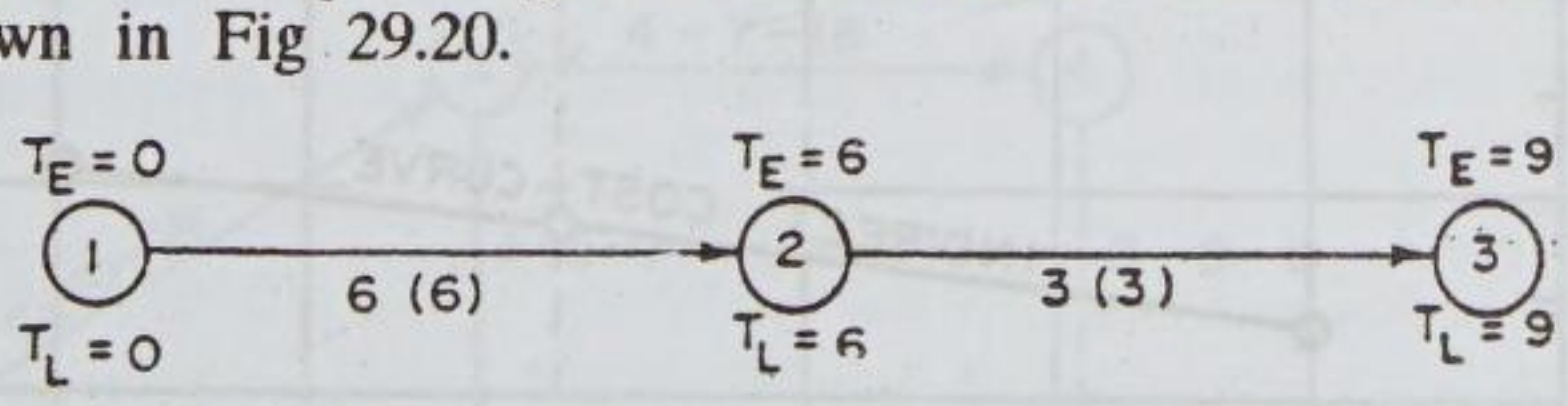


FIG. 29.20.

Step 5. *Total cost of project*

The total cost of the project, for any duration, is obtained by adding the indirect costs overheads to the corresponding direct c›sts. The values are tabulated in Table 29.5.

TABLE 29.5

Duration → (days)	*14 Normal*	*12*	*9*
Diret Cost	13000	13500	15000
Indirect cost	4200	3600	2700
Total cost	17200	17100	17700

Step 6. *Cost-duration curves.* Fig. 29.21 shows the cost-time curves for direct cost, indirect cost and total cost. From Table 29.5 as well as Fig. 29.21, it is evident that total cost is *minimum* for a project-duration of 12 days. Thus the *optimum duration* of the project is **12 days** and *minimum cost* corresponding to it is **Rs. 17100.**

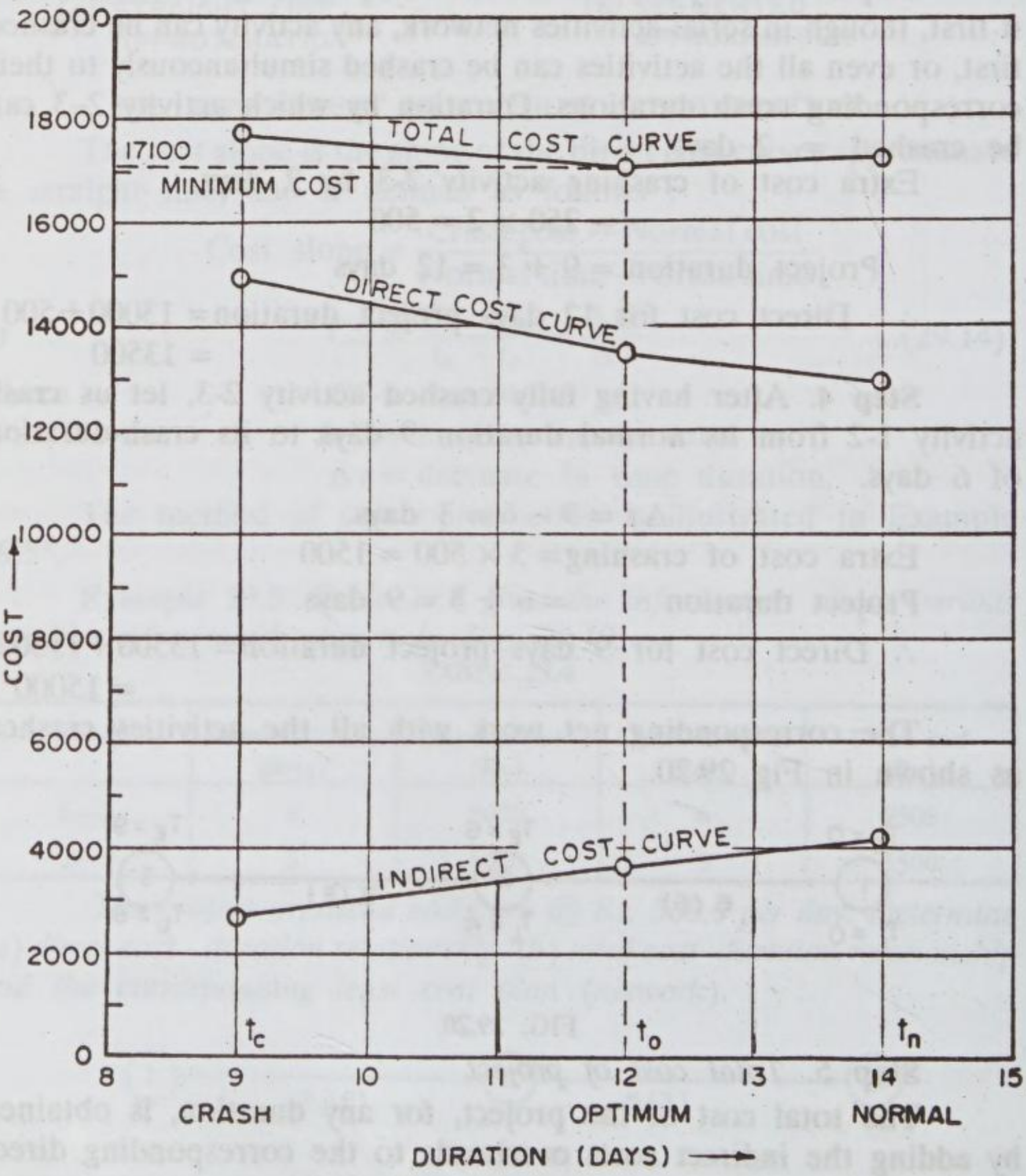


FIG. 29.21. COST-DURATION CURVES.

PROBLEMS

1. What is a Gantt bar chart ? Explain, with the help of a suitable example, the method of preparing a bar chart.

2. What is a milestone chart ? How does it differ from a bar chart? How can a milestone chart be developed into a network?

3. What are the short comings of bar charts ? How are these removed ?

4. A project consists of 8 activities *A, B, C, D, E, F, G* and *H* with their times of completion as follows :

Activities	*Duration (weeks)*
A	2
B	4
C	2
D	4
E	6
F	4
G	5
H	4

The precedence relationships are as follows :
A and *B* can be performed in parallel.
C and *D* cannot start until *A* is complete.
E cannot start until half the work of activity *C* is complete.
F can start only after activity *D* is complete
G succeeds *C*
H is the last activity, which should succeed *E*

(*a*) Draw the bar chart
(*b*) What is the total time of completion of the project ?
(*c*) If there is increase of 2 weeks in time of completion of activity A what will be the corresponding increase in the total time of the completion of the project ?

5. (*a*) Define an 'event' and an 'activity'. Differentiate between the two. (*b*) Define (*i*) head event (*ii*) tail event (*iii*) successor event (*iv*) predecessor event.

6. Differentiate clearly between PERT and CPM network methods.

7. The PERT network for a certain project is shown in Fig. 29.22. Determine the expected time for each path. Which path is critical ?

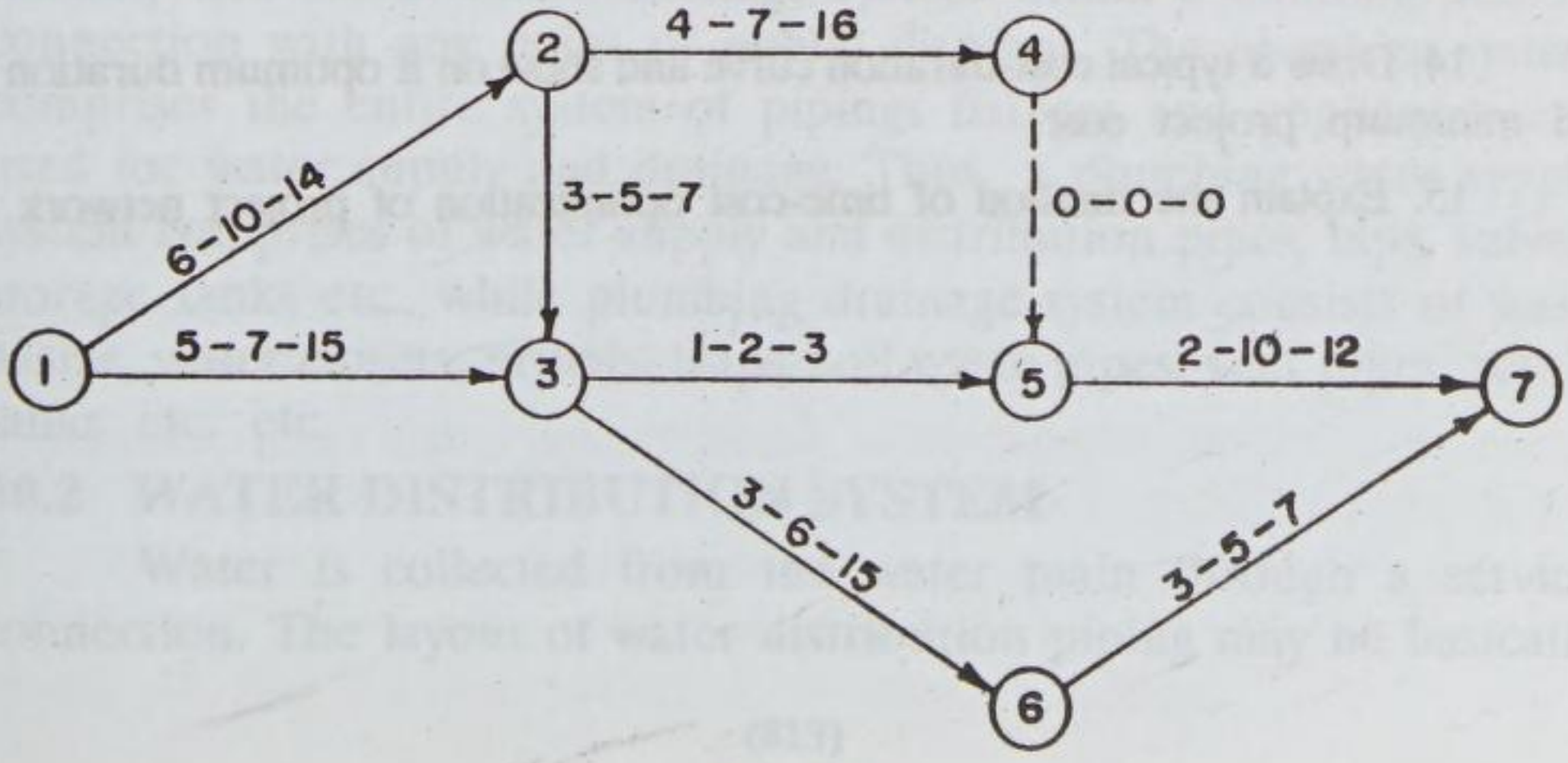


FIG. 29.22.

8. Define 'earliest event time' and 'latest occurrence event time'. How are these determined ? Explain the tabular form for determining these.

9. What do you understand by 'earliest start time' and 'latest start time of an activity'? How are these determined ?

10. Define 'latest start time' and 'latest finish time'. How are these determined ?

11. What do you understand by total float ? How is it determined? what is its importance in network planning ?

12. What do you understand by critical path ? How is it determined ?

13. The network of a certain project is shown in Fig. 29.23 with the estimated durations of various activities. Determine the following :

(*a*) Earliest event time and latest event time.

(*b*) Earliest and latest start and finish times of each activity.

(*c*) Total and free floats for each activity.

(*d*) Critical path for the network.

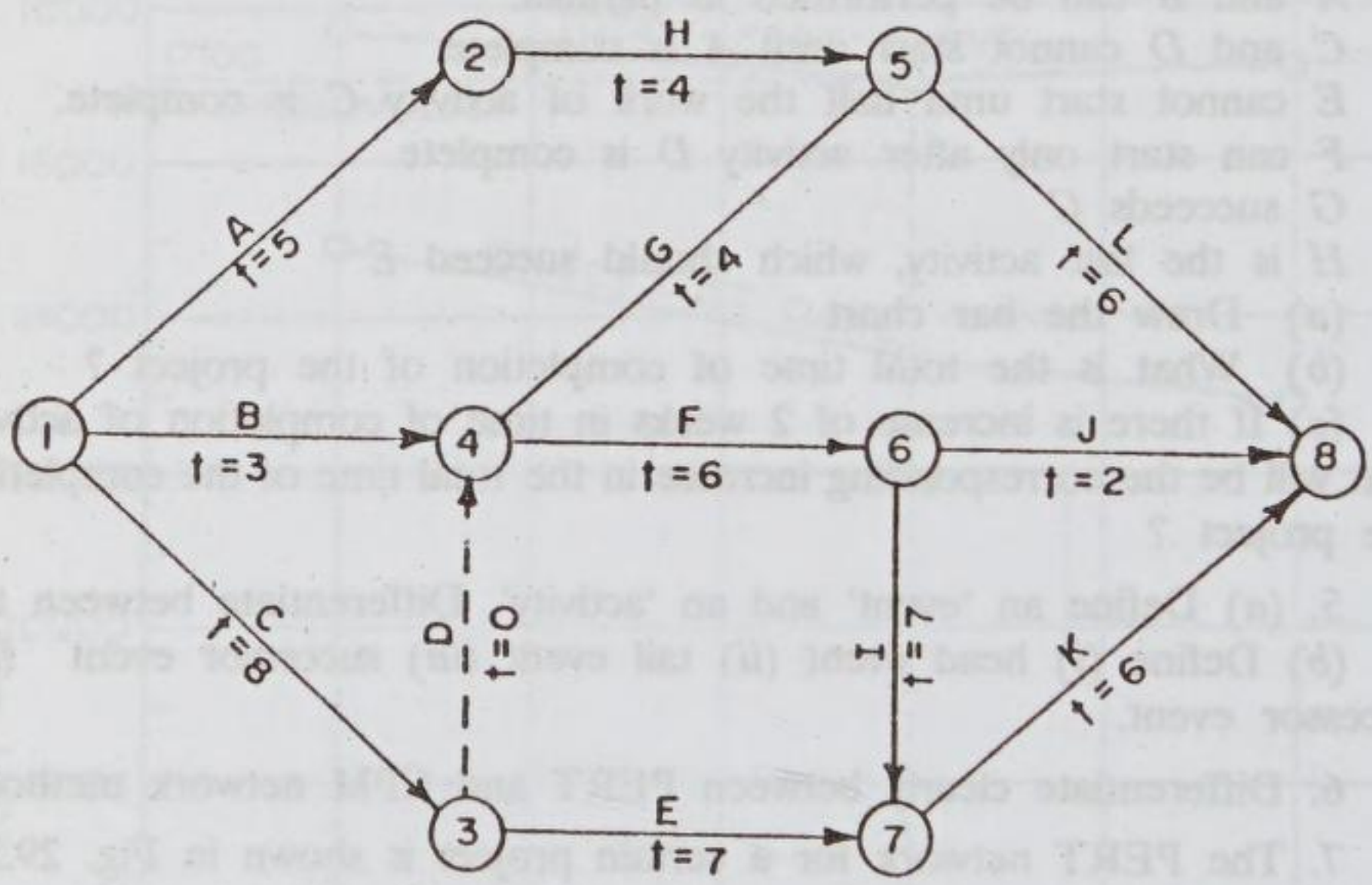


FIG. 29.23

14. Draw a typical cost-duration curve and show on it optimum duration and minimum project cost.

15. Explain the method of time-cost optimization of project network.

30

Water Supply and Drainage

30.1. INTRODUCTION : PLUMBING SERVICES

Public water supply systems follow the sequence of water collection from source of supply, conveyance to treatment plants, treatment including disinfection and, finally, its distribution. Ultimately, water is distributed for various consumption purposes in a building through internal water distribution system. In every building, adequate quantity of water should be available at required locations, to meet various needs of the occupants. Before designing a building, water needs must be properly computed. The transmission of water within the building is carried through pipes which may run either underground or above ground. The water so supplied ultimately gets converted into *waste water* , which has to be properly *drained* . Various fittings used for use of water, and for drainge of used water are commonly known as *sanitary fittings*.

The services like water supply, drainage, sanitation etc. are sometimes known as *plumbing services*. *Plumbing* is a general term indicating the practice, materials and fixtures used in the installation or maintenance of all piping, fixtures, appliances and other appertenances used in connection with water supply system as well as sanitary and storm water drainage system within a building and its connection with any point of public disposal. The *plumbing system* comprises the entire system of pipings fixtures and appliances etc. used for water supply and drainage. Thus a plumbing water sypply system comprises of water supply and distribution pipes, taps, valves, storage tanks etc., while plumbing drainage system consists of wash basins, water closets, urinals, traps, soil waste pipes, vent pipes, septic tanks etc. etc.

30.2 WATER DISTRIBUTION SYSTEM

Water is collected from the water main through a service connection. The layout of water distribution piping may be basically

a horizontal or vertical arrangement of limited height and in which underground mains *under pressure* supply water to the fixture inlets. Such a system is known as unfeed system. Alternatively, water is first collected in underground tank (known as suction tank) and then it is pumped to elevated storage tanks, usually situated at the top of the building. From these elevated storage tanks, water can flow down and feed the fixtures.

General considerations

While laying out the pipe lines, the following considerations should be kept in mind :

1. The lines should be so laid that there is no risk involved in the contaminating of water supply. For this, following three things are necessary :

(*a*) There is no cross-connection any where between a pipe carrying possible water and the pipe carrying used or waste water.

(*b*) There should be no back flow from any cistern or appliance towards the source of supply.

(*c*) Water supply pipes and waste water pipes (drainage pipes) should not be laid very close to each other.

2. The pipe line should be properly protected against any damage. To achieve this, underground pipe line should be enclosed in a cement mortar *bata* so that its rusting by soil bacteria is prevented, and also it should have earth cover of atleast 60 cm. When pipe line is laid above ground, it should run clear of water. Also, when it crosses a wall, it should be contained in suitable sleeve for the entire length of the crossing.

3. In the unfeed system, pipe should carry water under adequate pressure. For this, lay out of pipe should be simple and direct as far as possible. The pipes should be laid out as straight as possible.

Estimation of water requirements

For *residential buildings*, Indian Standard recommends that a water requirement of 135 litres per head per day may be assumed. Out of this, 90 litres may be taken for domestic purposes while the balance 45 litres are taken for flushing requirements.

The requirements of water for buildings other than residence may be found from Table 30.1.

TABLE 30.1. WATER REQUIREMENTS FOR BUILDINGS OTHER THAN RESIDENCES

Type of building	*Rate per head per day in litres*
1. Factories, where bath rooms are required to be provided.	45
2. Factories where no bath rooms are required to be provided.	30
3. Hospital (including laundry) per bed.	
(*i*) Number of beds not exceeding 100	340
(*ii*) Number of beds exceeding 100	450
4. Nurses homes and medical quarters.	135
5. Hostels	135
6. Hotels (per bed)	180
7. Offices	45
8. Restaurants (per seat)	70
9. Cinemas, concert halls and theatres (per seat).	15
10 Schools	
(*a*) Day Schools	45
(*b*) Boarding Schools	135

30.3. MATERIAL FOR SERVICE PIPES

The pipe leading from the distribution main of the municipal water supply to the plumbing system of the house is known as the *service main* . The following materials are commonly used for service pipes :

(*i*) Copper pipe or brass pipe
(*ii*) Galvanised iron, either lined or unlined
(*iii*) Load pipe, either lined or unlined
(*iv*) Polythene pipe.

1. Copper tubing. Copper tubing is non-corrosive with most waters. It is used extensively in better grade houses and where ground water is highly corrosive to steel pipe. It has considerable strength, reasonable ductility, and is obtainable in long lengths. There are two types of copper water service pipes : *heavy gauge* and *light guage*. The former can be threaded and is used for high pressure work in industrial layouts. For general purpose work where pressure does not exceed 1.5 kg/cm^2, and for interval domestic work, light gauge tube is used. For underground work, as for the pipe from the main to the building, special copper tube is used having a heavier guage. The copper tube can be attached to the main without the use of conventional goose neck, the flared end of the tubing being connected directly to the corporation cock without threading.

2. G.I. pipes. These are used where water is suitable. In India, these are extensively used because of their low cost in comparison to copper tubes. Galvanised iron or steel tubes corrode more readily in soft and acid waters and are not so easily manipulated as copper, although they are stronger and can be used in hard water area where they withstand the hammering needed to remove the scale.

3. Lead pipe. Lead pipe has the following advantages (*i*) highly resistant to corrosion (*ii*) highly flexible and (*iii*) high hydraulic coefficient of flow. However, lead has cumulative poisoning effect, specially when it goes into solution. Due to this, it is not preferred.

4. Polythene tube. These are being increasingly used internally and externally for cold water service pipes only. Plastic pipe has the following advantages over metal piping. (*i*) it is lower in cost (*ii*) it is non corrosive (*iii*) it is light in weight (*iv*) it can be installed with ordinary tools, (*v*) it does not require threading (*iv*) it is more resistant to bacterial scale and (*vii*) it has some insulating value. However, these are not useful where temperature is high.

30.4. SERVICE CONNECTION

A service connection is primarily a connection from the distribution system to the consumer. A consumer may be a single house, an apartment house, a planned block development or a water district buying water 'wholesale'. A connection for a single house will normally involve tapping the main while it is under pressure and installing a corporation cock.

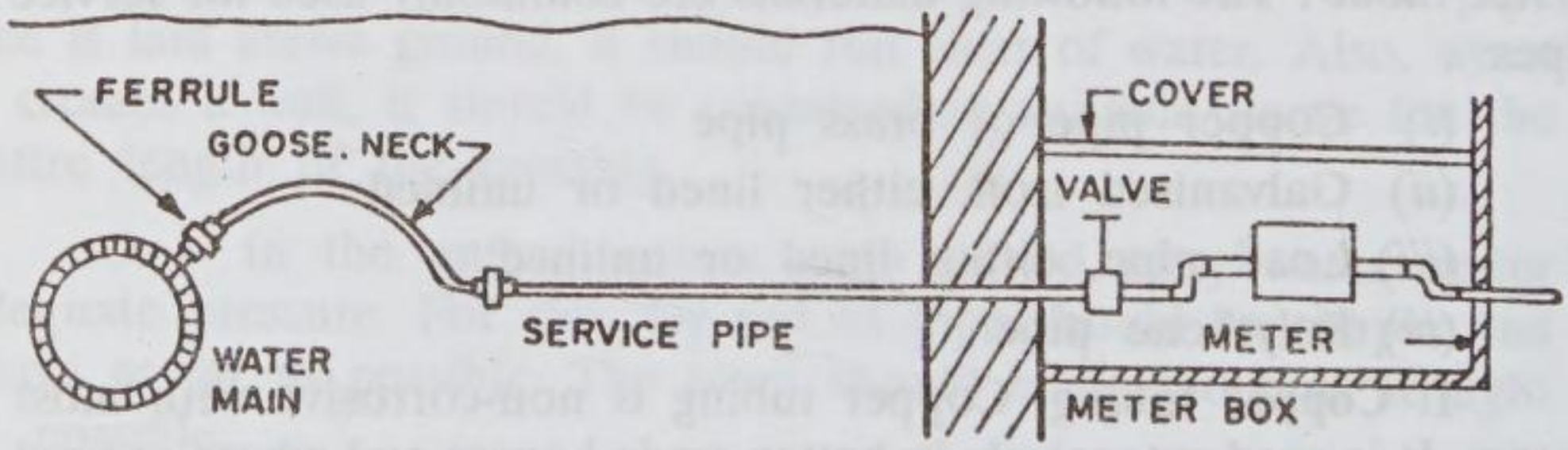


FIG. 30.1. SERVICE CONNECTION.

A domestic service connection includes the following components :

1. Brass or bronze ferrule. Ferrule is a right angled sleeve made of brass or bronze. For a typical service connection to a house, a ferrule is inserted in the main, most usually an 'under pressure' connection which can be inserted without shutting down the main.

2. Goose neck. This consists of a 40 to 50 cm long curved piece of flexible pipe made of brass, copper or lead. The goose

neck prevents the breaking of the main service pipe due to movement that takes place between the water main and the service pipe, thus providing flexibility of the junction.

3. Stop cock or curb valve. It is installed in a suitable chamber with cover, to close down the supply, for repairs of the plumbing system.

4. Main service pipe. It may be of various materials discussed in Sec. 30.3. Its diameter may very from 12 mm to 40 mm.

5. Water meter. It is also installed in a suitable chamber with cover. It measures the quantity of water used by the consumer.

30.5. SIZE OF SERVICE PIPES

The size of the service pipes are determined on the basis of the following : (*i*) the minimum pressure in the distribution main at the proposed point of connection, (*ii*) length of service pipe required, (*iii*) elevation of the highest point of delivery above the distribution mains, (*iv*) number and types of plumbing fixtures in the building, and (*v*) maximum rate of flow required. The maximum rate of flow may be estimated by considering what the average householder expects at his plumbing fixtures. Guidance may be taken from Table 30.2.

TABLE 30.2

Plumbing Fixture	*Flow Required (litres/min.)*	
	Good	*Reasonable*
Kitchen Tap	10	7
Bath Tap (cold)	25	15
W.C. flushing cistern	10	7

If it is assumed that one of each of these is in operation simultaneously, a maximum demand rate of 29 to 45 litres/min. is obtained. Knowing the rate of flow and the desired velocity, pipe diameter can be found. Alternatively, the diameter of service pipe can also be fixed on the basis of occupants in the house, as given in Table 30.3.

TABLE 30.3

No. of occupants	*4*	*8*	*24*	*60*
Dia. of service pipe (mm)	12.5	20	25	30

30.6. WATER METER

Water meter is normally used for measuring flows to domestic buildings. A water meter should possess the following characteristics:

1. It should accurately measure and register both small and large flows.

2. It should be easy to maintain and repair. Spare parts should be readily available.

3. It should have good capacity with reasonable head loss.

4. It should be capable of working at all pressures efficiently.

5. It should be durable. Its parts should not be affected by chemicals used for purification and the impurities in water.

6. It should be rugged.

7. It should prevent back flow passing through it and should not be liable to clogging.

8. It should have low cost.

Meters used on water distribution systems may be of two types :

(*a*) Inferential or velocity meters, and

(*b*) Positive or displacement meters.

Inferential meter. It measures the velocity of flow across a cross-section whose area is known. They are used only for high flows. Common examples of this type of meters are the rotary and the turbine meters.

Displacement meter. These are primarily used for relatively low flows as for the residential buildings. In this meter, the quantity of water actually passing through it is measured by filling and emptying

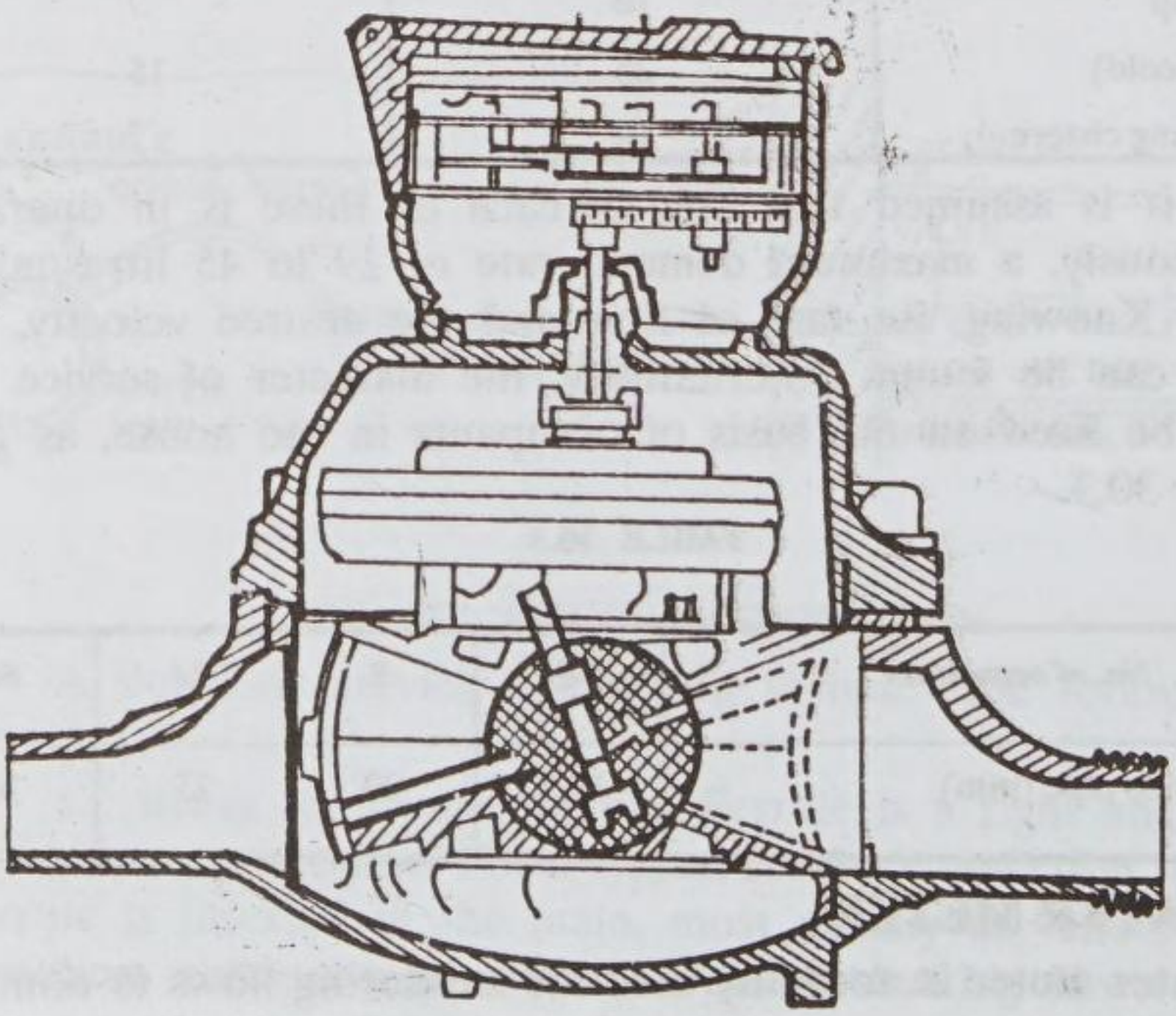


FIG. 30.2 NUTATING-DISC METER.

the chamber of known capacity. Types of displacement meters in use include resiprocating, rotary, oscillating and nutating disc meters, depending upon the motion of the moving part in the measuring device.

Disc meter, shown in Fig. 30.2 consists of a disc of hard rubber placed inside a chamber which is provided with inlets and outlets. The water entering the chamber oscillates the disc about its centre with a spiral motion. The oscillations imparted by one complete filling and emptying are recorded by the disc meter in terms of volume of water.

30.7. VALVES

For domestic water supply, two types of valves are commonly used :

(*a*) Globe valve, and
(*b*) Gate valve.

Globe valve. It is used in pipe lines for convenience in manually closing the pipes to control the flow of water. Fig. 30.3 shows the section of a globe valve. It should be installed with water pressure under the valve seat as shown. The globe valve has advantage of quicker opening and closing, of longer life and of being more easily repaired.

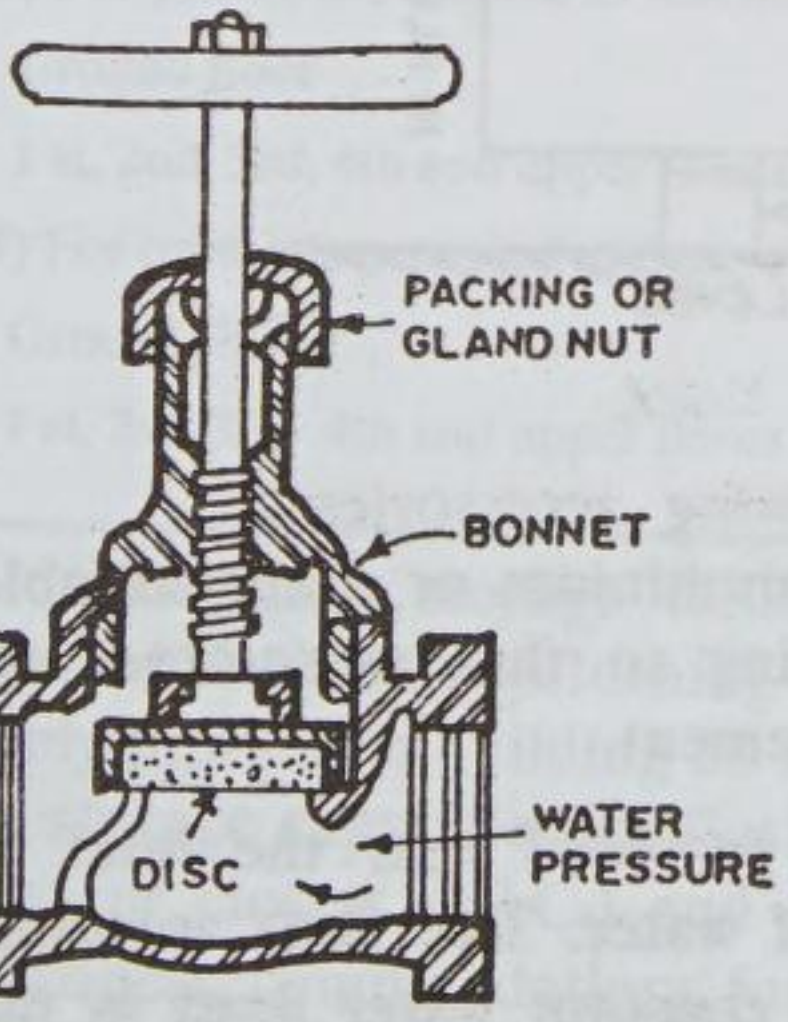


FIG. 30.3. GLOBE VALVE.

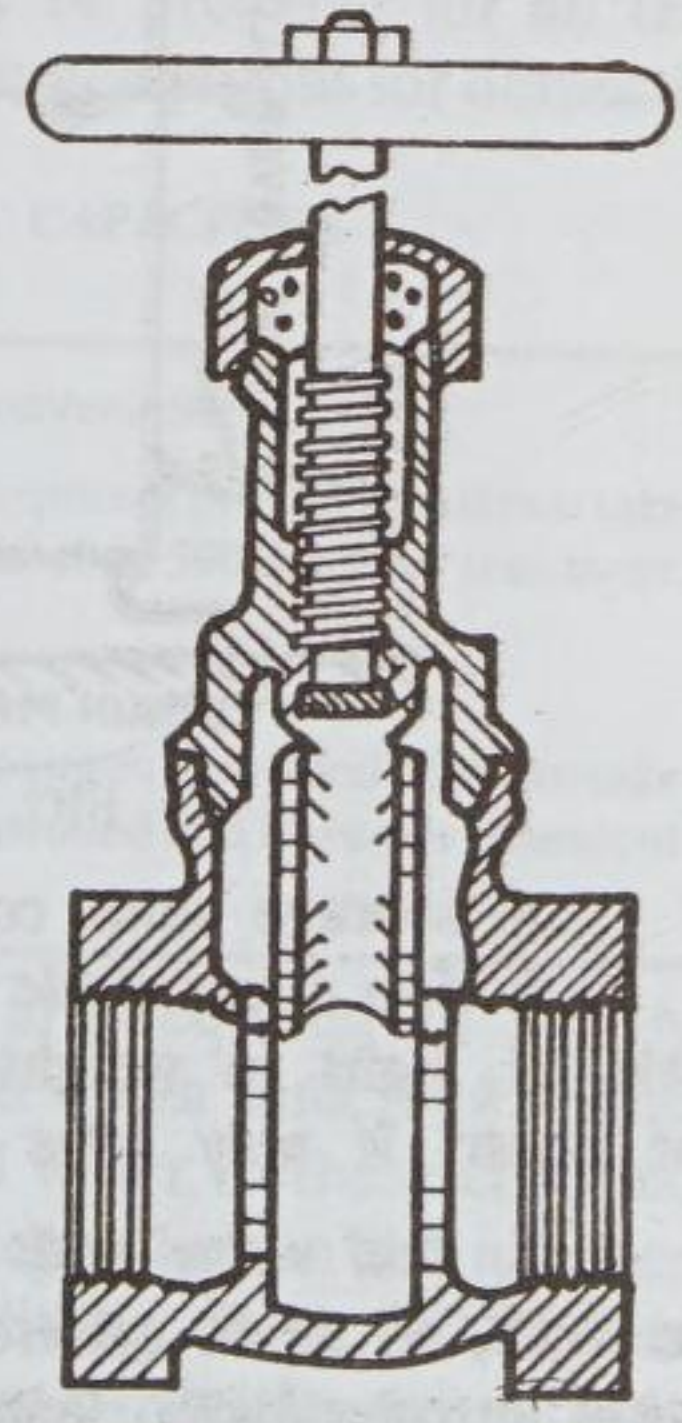


FIG. 30.4 GATE VALVE.

Gate valve. Gate valve is used in pipe lines for convenience in manually closing the pipes. The gate valve has an advantage over the globe valve in that it offers less resistance to flow. It is therefore used in preference to the globe valve where resistance to the flow of water is to be kept at a minimum. It also controls the flow equally well from either direction.

30.8 STORAGE TANKS

The water supply to a building may either be continuous or intermittant. Even in the case of continuous supply in the mains, the pressure of water may not be sufficient to rise to all the floors of the building. In either case, storage tanks are required. The storage tanks may be situated either at the ground level, or at the roof level or at both the levels. If the pressure of water is sufficient to rise to the roof level, storage tank is provided only at the roof level, so as to store water because of intermittant supplies. If the pressure of water is not sufficient, water is first stored at the ground tank from where it is pumped to the top storage tank.

A storage tank is made of the following materials : (*i*) mild steel pressed plates (*ii*) reinforced concrete, or (*iii*) stone or brick masonry. Fig. 30.5 shows a typical section of a storage tank.

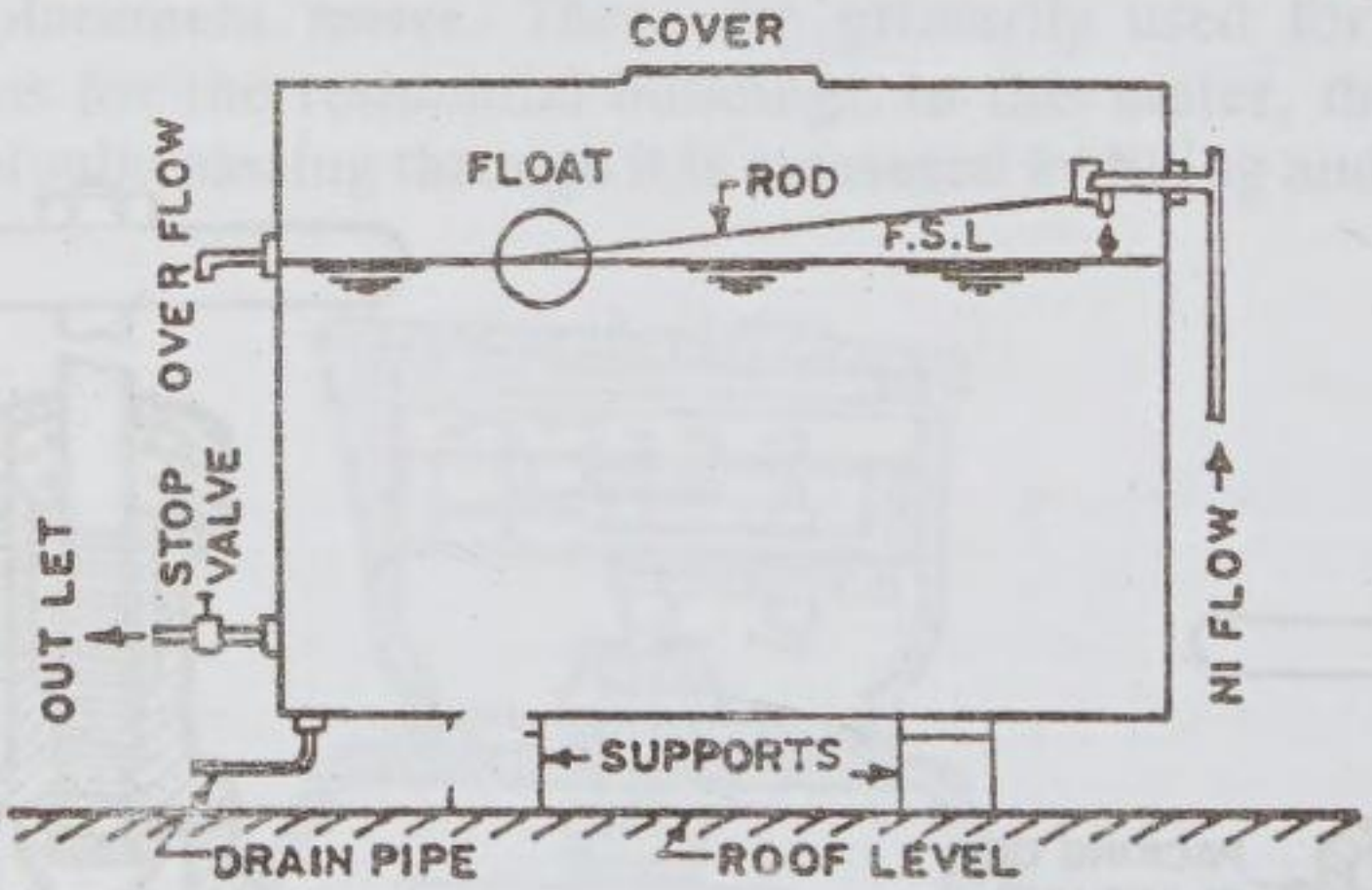


FIG. 30.5. STORAGE TANK

A storage tank consists of following accessories :

1. *Top cover* made of mild steel, aluminium or other suitable material, light in weight and tight fitting so that mosquitoes do not enter. It may have locking arrangement.

2. *Ball valve with float* which is provided near the inlet to the tank, so as to control the inflow of water. The float assembly works automatically, thus maintaining a constant water level in the tank. When the water level falls down, the float moves down thus opening the inlet valve. When the level in tank reaches the desired

full supply level (F.S.L.), the float rises up thus closing the inlet valve.

3. *Over flow pipe* which is set about 2.5 cm above F.S.L. In case the float assembly fails, the inflow is not cutoff and the water entering the tank overflows through this pipe.

4. *Supply pipe or inlet pipe*, admitting water into the tank.

5. *Outlet pipe with stop valve* set about 2.5 to 5 cm above the bottom of the tank, for cutting off the supply to down tank pipes.

6. *Drain pipe or scour pipe* for cleaning the tank periodically.

Capacity of storage tank

The capacity of storage tank depends upon the following factors:

(*i*) Supply hours from the mains, with sufficient pressure.

(*ii*) Frequency with which the tank can be refilled during 24 hours.

(*iii*) Rate and regularity of supply.

In the case of multi storeyed building, the first two or three storeys may be fed directly from the water mains while upper storeys are fed directly from the storage tanks. However, if the supply is either intermittant or irregular, storage may be provided for all the storeys. Table 30.4 give Indian Standard Recommendations for domestic storage capacities.

TABLE 30.4. DOMESTIC STORAGE CAPACITIES

(*A*) For premises occupied as tenements with common convenience	
1. Ground floor	No storage required, provided no down take fittings are installed 500 litres per tenement.
2. 1 st, 2nd, 3rd, 4th and upper floors	
(*B*) For premises occupied as flats or blocks :	
1. Ground flkoor	No storage required, provided no down take fittings are installed 800 litres per tenement
1. 1 st, 2nd 3rd, 4th and upper floors	

Flushing storage tank. These tanks are required to supply water to the various flushing cisterns through down tank pipes. The supply pipes of a building do not directly feed water to these cisterns. The storage capacities for flushing purposes depend upon the number of water closets (W.C.) and urinals in a building. Table 30.5 gives I.S. code recommendations for flushing storage capacities for various types of buildings.

TABLE 30.5. FLUSHING STORAGE CAPACITIES

Classification of building	*Storage Capacity*
1. For tenements having common conveniences.	900 litres net per W.C. seat.
2. For residential premises other than tenements having common conveniences.	270 litres net for each W.C. seat and 180 litres for each additional seat in the same flat.
3. For factories and workshops.	900 litres per W.C. seat and 180 litres per urinal seat.
4. For cinemas, public assembly hall etc.	900 litres per W.C. seat and 360 litres per urinal seat.

30.9. HOUSE DRAINAGE : GENERAL PRINCIPLES

The arrangement provided in a house or building, for collecting and conveying waste water through drain pipes, by gravity, to join either a public sewer or a domestic septic tank, is termed as *house drainage* or building drainage.

Aims of house drainage

House drainage is provided

(*i*) to maintain healthy conditions in the building

(*ii*) to dispose off waste water as early and quickly as possible

(*iii*) to avoid the entry of foul gases from the sewer or the septic tank

(*iv*) to facilitate quick removal of foul matter (*e.g.* human excreta)

(*v*) To collect and remove waste matters systematically

Principles of house drainage

The following principles are adopted for the efficient drainage system :

1. The lavatory blocks should be so located that the length of drainage line is minimum. In the case of multistoreyed building they should be located one above the other. At least one wall of the lavatory block should be an outside wall, to facilitate the fixing of soil and vent pipes.

2. The drainage pipes should be laid by the side of the building rather than below the building.

3. All the drains should be aligned straight between successive inspection chambers. All sharp bends and junctions should be avoided except through chambers.

4. The slope of the drains should be sufficient to develop self cleansing velocity.

5. The size of drain should be sufficient, so that flooding of the drain does not take place while handling the maximum discharge.

6. The drainage system should contain enough number of traps at suitable locations.

7. The house drain should be disconnected to the public sewer by the provision of an intercepting trap. This will avoid the entry of foul gases from entering the house drainage system. It should be seen that the public sewer is deeper than the housedrain.

8. Rain water pipes should drain out rain water directly into the street gutters from where it is carried to the storm water drain.

9. All the connections should be water tight.

10. The entire drainage system should be properly ventilated from the starting point to the final point of disposal. It should permit free circulation of air.

11. All the materials and fittings of the drainage system should be hard, strong and resistant to corrossove action. They should be non-absorbent type.

12. The entire system should be so designed that the possibilities of formation of air locks, siphonage, under deposits etc. are minimised.

30.10. PIPES AND TRAPS

Pipes. In a house drainage system, a pipe may have the following designations, depending upon the function it carries :

1. *Soil pipe.* A soil pipe is a pipe through which human excreta flows.

2. *Waste pipe.* It is a pipe which carries only the liquid waste. It does not carry human excreta.

3. *Vent pipe.* It is a pipe which is provided for the purpose of the ventilation of the system. A vent pipe is open at top and bottom, to facilitate exit of foul gases. It is carried at least 1 m higher than the roof level.

4. *Rain water pipe.* It is a pipe which carries only the rain water.

5. *Anti-siphonage pipe.* It is pipe which is installed in the house drainage to preserve the water seal of traps.

The following sizes of pipes are commonly used in house drainage :

Soil pipe	: 100 mm
Waste pipe : horizontal	: 30 to 50 mm
Waste pipe : vertical	: 75 mm
Rain water pipe	: 75 mm
Vent pipe	: 50 mm

Anti-siphonage pipe :
(*i*) Connecting soil pipe : 50 mm
(*ii*) Connecting waste pipe : 40 mm

Traps

A traps is a depressed or bent fitting which, when provided in a drainage system, always remains full of water, thus maintaining a *water seal.* It prevents the passage of foul air or gas through it, though it allows the sewage or waste water to flow through it. The depth of water seal is the vertical distance between the crown and dip of a trap (Fig. 30.6). The depth of water seal represents its *strength* or effectiveness. Greater the depth of water seal, more effective is the trap. The depth of water seal varies from 25 mm to 75 mm.

Causes of breaking of seal . Water seal may break due to the following reasons :

(*i*) faulty joints
(*ii*) crack in the bottom of seal
(*iii*) creation of partial vacuum in the sewer fittings
(*iv*) increase in the pressure of sewer gases, and
(*v*) non-use for a prolonged period.

The breaking of the water seal can be prevented by (*i*) connecting the portion between the soil pipe and trap by a vent pipe, and (*ii*) use of anti-siphonage pipe in the building.

Characteristics of traps . A trap should possess the following characteristics :

1. It should possess adequate water seal at all times, to fulfill the purpose of its installation. However, it should retain minimum quantity of water for this purpose.
2. It should be of non-absorbent material.
3. It should be free from any inside projections, angles or contractions, so that flow is not obstructed or retarded.
4. It should be simple in construction, cheap and readily available.
5. It should be self cleansing.
6. It should be provided with suitable access for cleaning.
7. Its internal and external surfaces should have smooth finish so that dirt etc. does not stick to it.

Classification of traps

Traps are classified as follows :

(*a*) *Classification according to shape* (Fig. 30.6)

(*i*) *P-Trap* (Fig. 30.6 *a*). This resembles the shape of letter *P*, in which the legs are at right angles to each other.

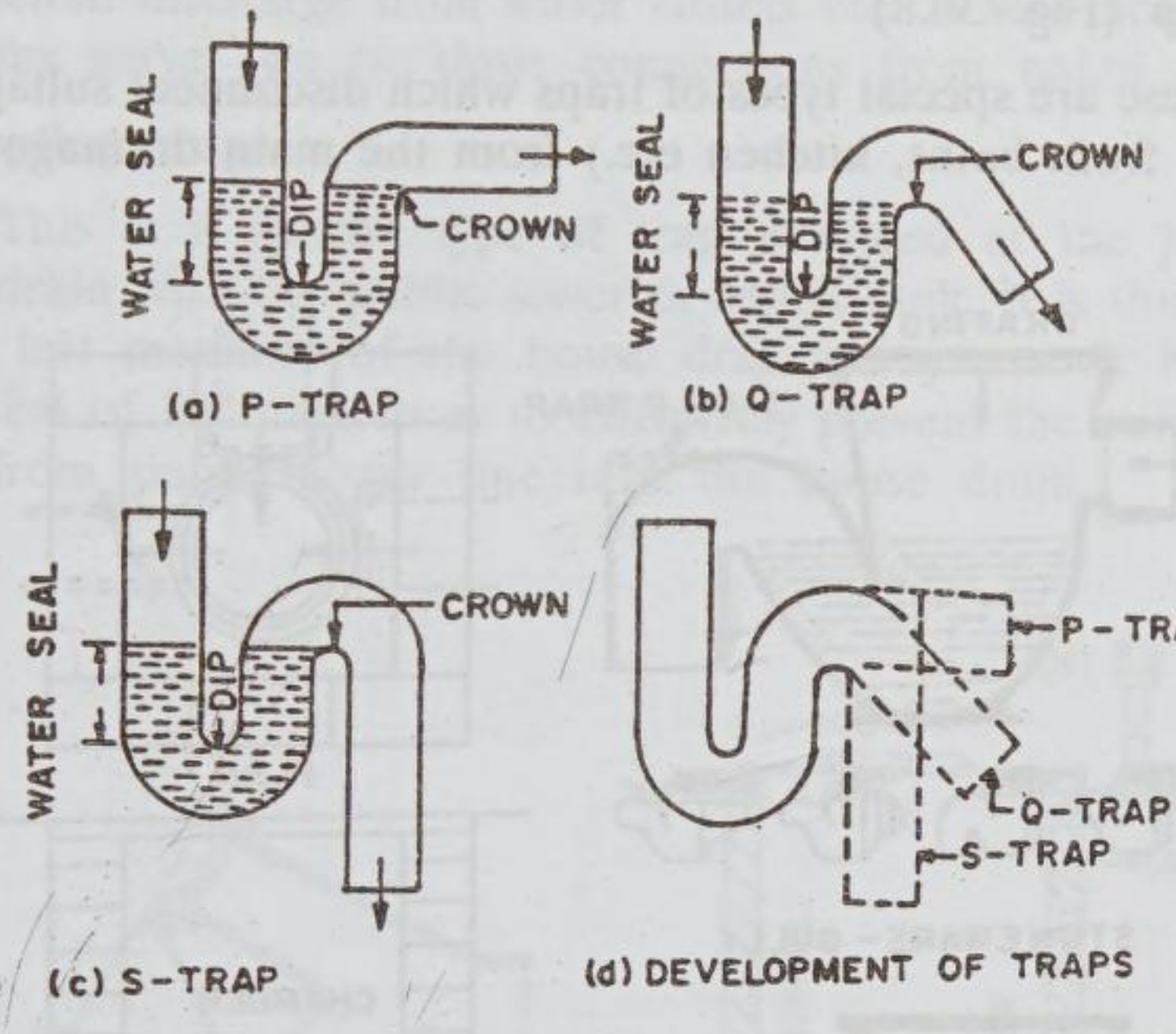


FIG. 30.6. TRAPS ACCORDING TO SHAPES.

(*ii*) *Q-trap* or *half-S-trap* (Fig. 30.6 *b*). This resembles the shape of letter *Q*, in which the two legs meet at an angle other than a right angle.

(*iii*) *S-trap* (Fig. 30.6 *c*). This resembles letter-*S*, in which both the legs are parallel to each other, discharging in the same direction. Fig. 30.6 (*d*) shows the development of all the three types of traps.

(*b*) *Classification according to use :*

(*i*) Floor trap or nahni trap

(*ii*) Gully trap

(*iii*) Intercepting trap.

Floor trap or Nahni trap

A floor trap, commonly known as a *nahni trap* is used to collect wash water from floors, kitchens and bath rooms. It forms the starting point of waste water floor. It is made of cast iron, with a gravity at top, to exclude entry of solid matter of big size. This cover can be removed to do frequent cleaning of the trap. These traps have small water seal.

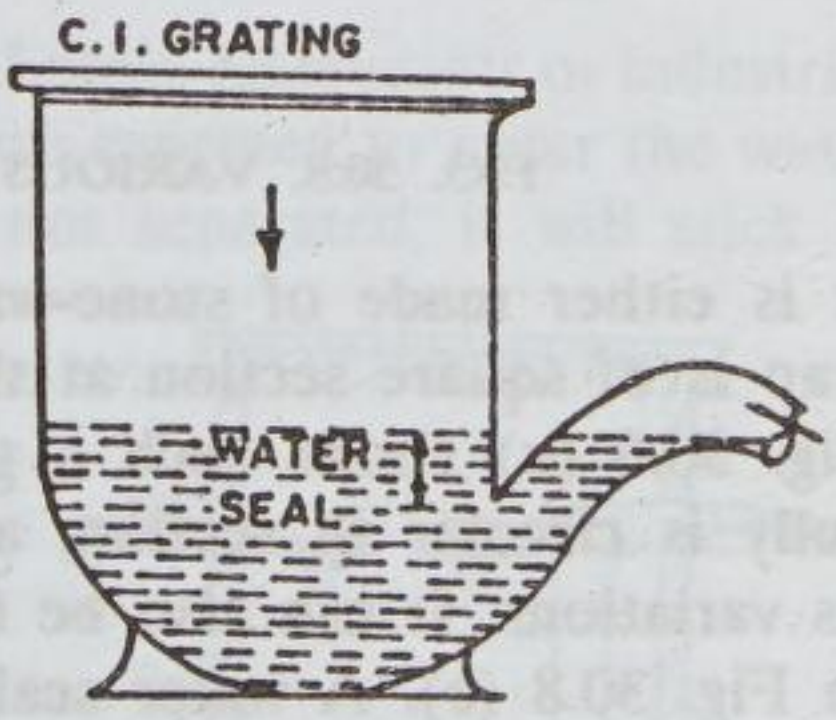


FIG. 30.7. FLOOR TRAP.

Gully trap (Fig. 30.8)

These are special types of traps which disconnect sullage drain (collected from baths, kitchen etc.) from the main drainage system.

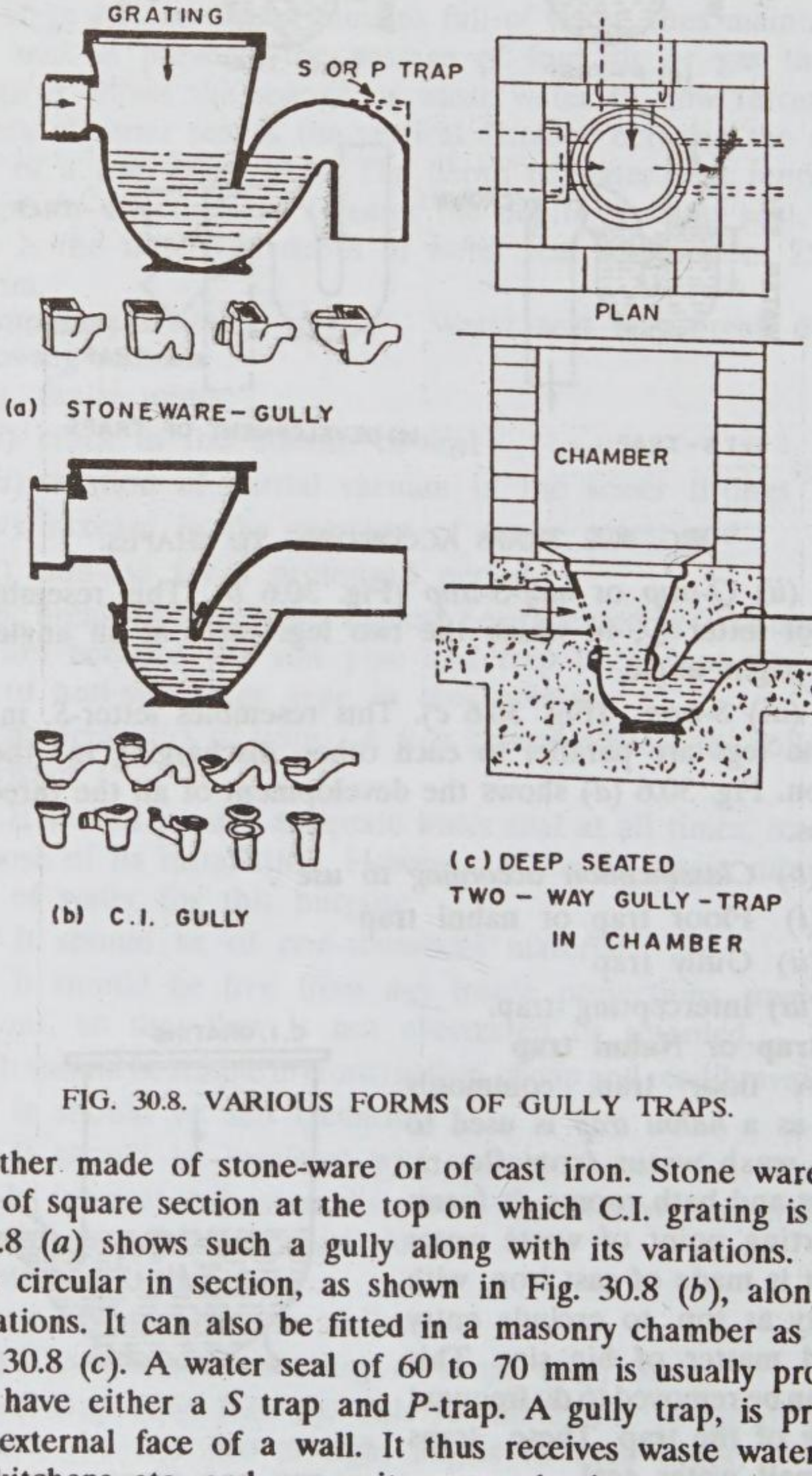


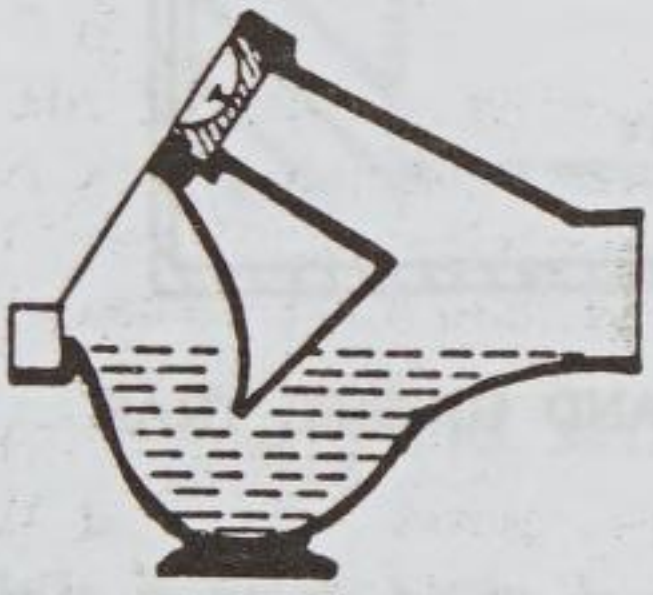
FIG. 30.8. VARIOUS FORMS OF GULLY TRAPS.

It is either made of stone-ware or of cast iron. Stone ware gully trap is of square section at the top on which C.I. grating is fitted. Fig. 30.8 (*a*) shows such a gully along with its variations. A C.I. gully is circular in section, as shown in Fig. 30.8 (*b*), along with its variations. It can also be fitted in a masonry chamber as shown in Fig. 30.8 (*c*). A water seal of 60 to 70 mm is usually provided. It may have either a *S* trap and *P*-trap. A gully trap, is provided at the external face of a wall. It thus receives waste water from baths, kitchens etc. and pass it on to the house drain carrying

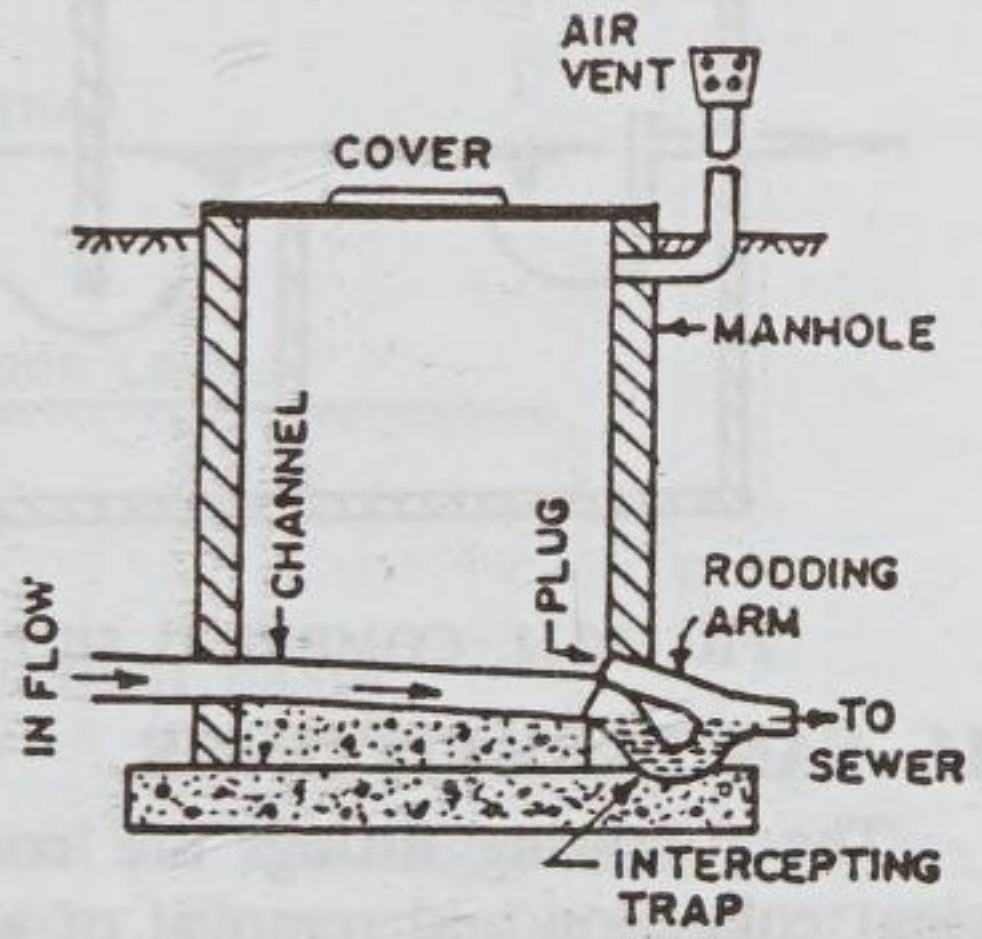
excremental discharge from water closets etc. A well designed gully trap may serve two or three connections from nahni traps.

Intercepting traps

This is a special type of trap provided at the junction of house drain with the public sewer or septic tank. It is thus provided in the last manhole of the house drainage system. It has a deep water seal of 100 mm, so as to effectively prevent the entry of sewer gases from public sewer line into the house drain.



(a) INTERCEPTING TRAP



(b) MANHOLE WITH INTERCEPTING TRAP

FIG. 30.9 INTERCEPTING TRAPS.

The trap has an opening at the top, called the *cleaning eye* or *rodding arm*, having a tight fitting plug, for frequent cleaning of the trap.

Grease traps

Such traps are used only in large hotels, restaurants or industries where large quantities of oily wastes are expected to enter the water flow. If the oily or greasy matter is not separated, it will stick to the building drainage system resulting in the formation of ugly scum and consequent obstruction to reaeration. A grease trap is either a masonry or cast iron chamber, with a bent pipe or Tee-pipe at the outlet end. Because of sudden increase in the area of flow at entry, the velocity of flow is reduced,

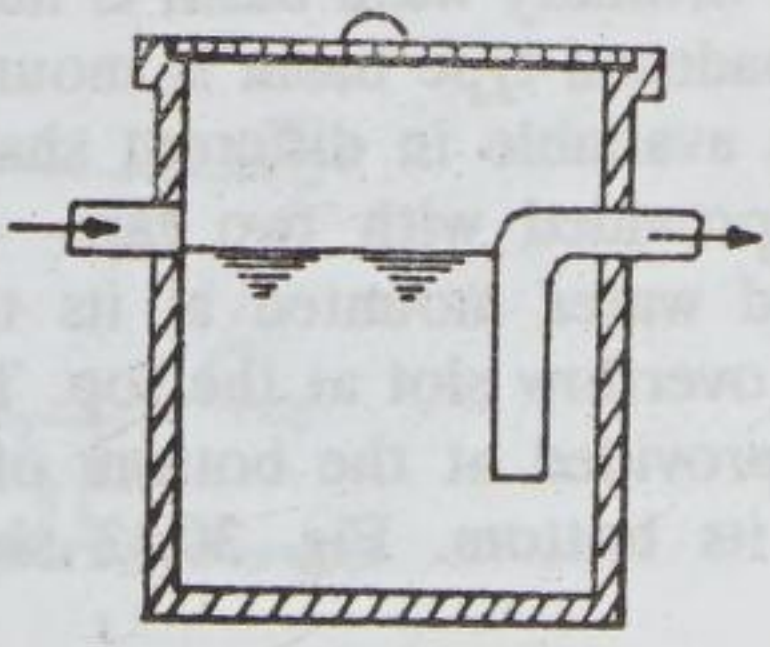


FIG. 30.10 GREASE TRAP

resulting in the separation of oily and greasy matter from the waste water. This greasy matter, floating on the top can be removed later.

Silt traps

If the water carries a lot of coarse particles of silt, sand etc., it is better to remove these before entering into the building drain. The silt particles normally enter the drain because of its use in washing the utensils. A silt trap is just similar to a grit chamber. Fig. 33.11 shows a combined silt and grease trap.

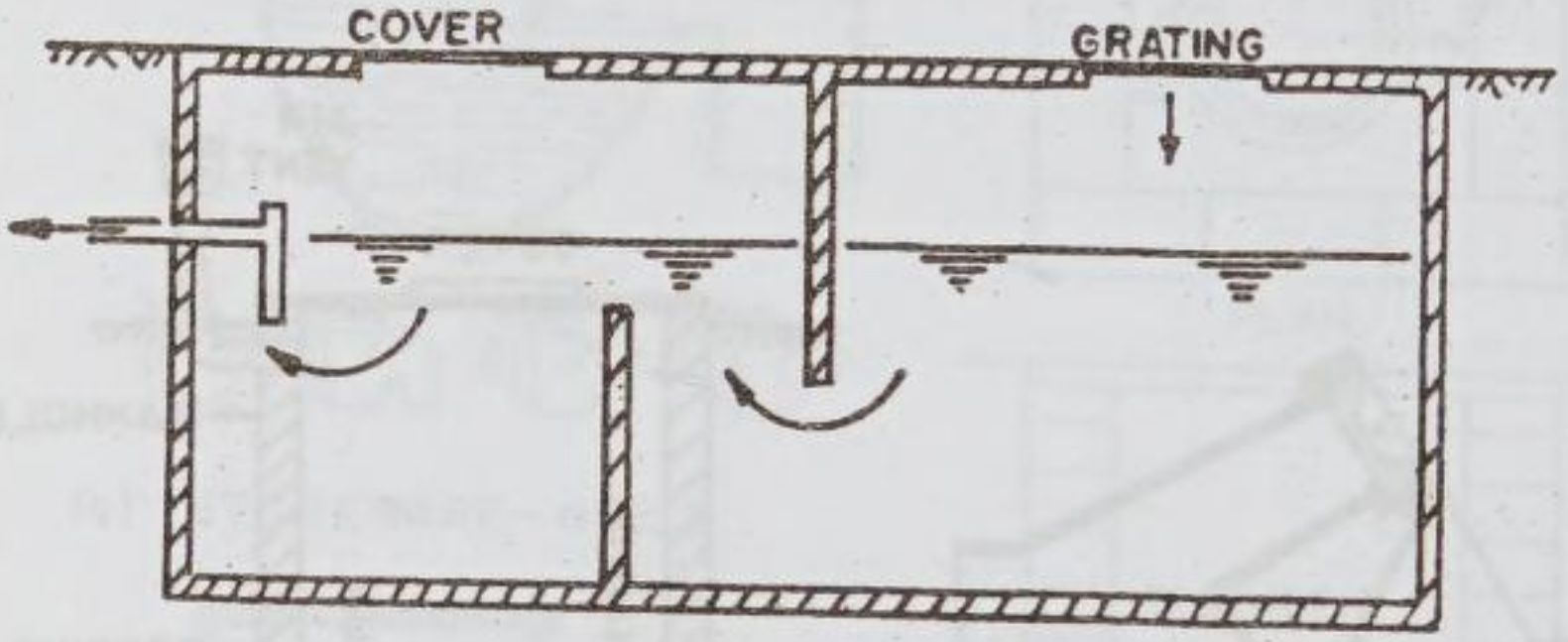


FIG. 30.11. COMBINED SILT AND GREASE TRAP.

30.11. SANITARY FITTINGS

The following fittings are commonly used in buildings, for efficient collection and removal of waste water to the house drain:

(*i*) Wash basins
(*ii*) Sinks
(*iii*) Bath tubs
(*iv*) Water closets
(*v*) Urinals
(*vi*) Flushing cisterns.

1. Wash Basin

Wash basins are usually made of pottery or porcelain ware. Sometimes, they are also made of porcelain enamelled cast iron, pressed steel or plastic, specially where number of users are more. An ordinary wash basin is mounted on brackets fixed on wall, while a padestal type basin is mounted on padestal rising from wall. They are available in different shapes and sizes. Normally, a wash basin is provided with two taps- one for hot water and the other for cold water mounted at its top. It has an oval shaped bowl, with an overflow slot at the top. The waste pipe with a mettallic strainer is provided at the bottom of the bowl. The waste pipe has a trap at its bottom. Fig. 30.12 shows a flat bottom wash basin.

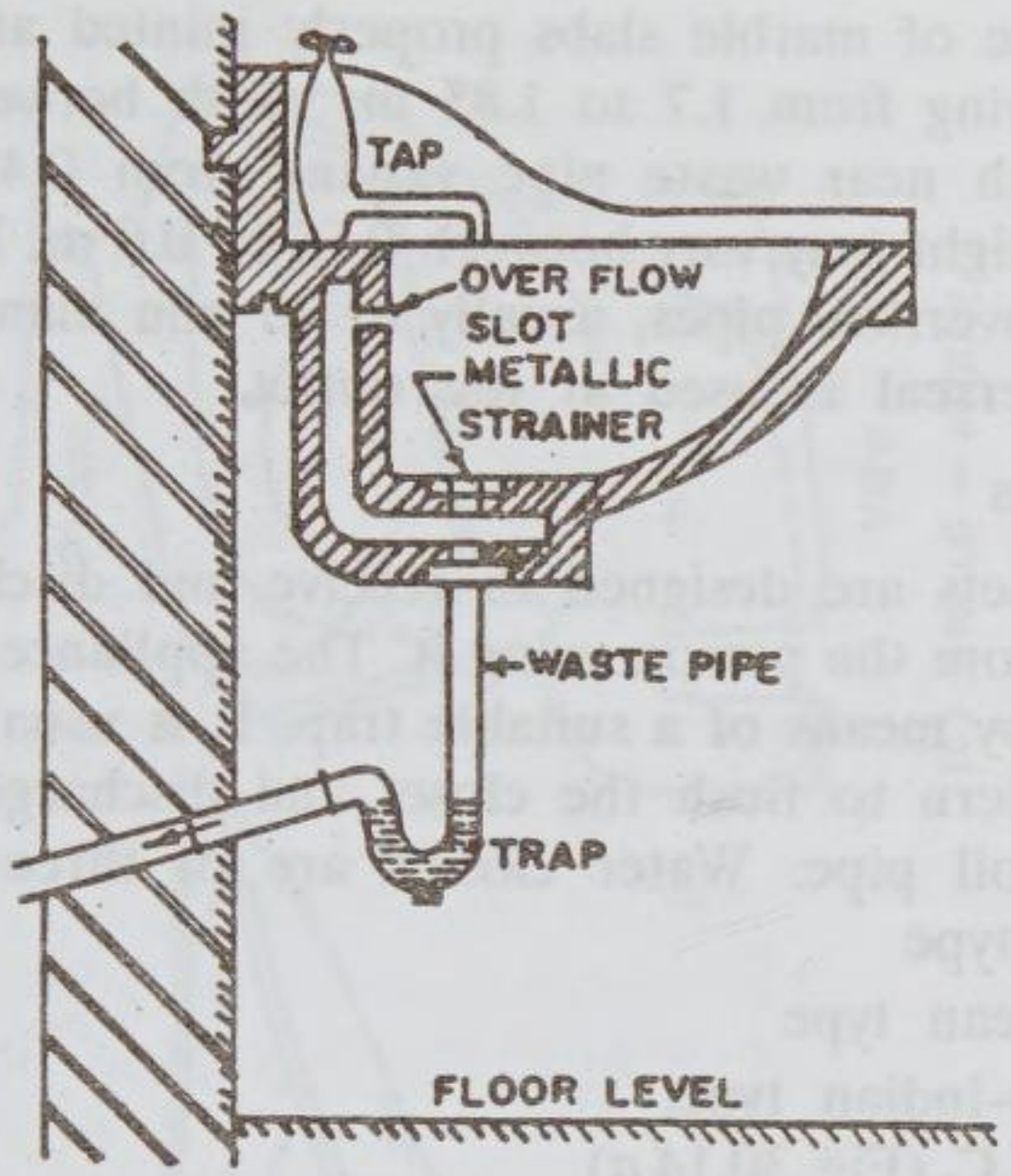


FIG. 30.12. WASH BASIN.

2. Sink.

While a wash basin is used for washing hands, face etc. a sink is used in kitchen or laboratory. These may be made of glazed fire clay, stainless steel, metal porcelain or enamelled pressed steel. They are manufactured in various sizes and shapes, though rectangular shape is quite common in kitchens. It may also have a drain board attached to it. A sink may also be constructed of cast-in-situ concrete, with suitable finishing surface such as marble, terrazo etc. The outlet pipe, provided with a grating of brass or nickel, may discharge over a floor trap or nahni trap.

3. Bath tub

Bath tubs, are usually made of iron or steel coated with enamel, enamelled porcelain or of plastic material. They may also be made of cast-in-situ concrete finished with marble chips or terrazzo, or

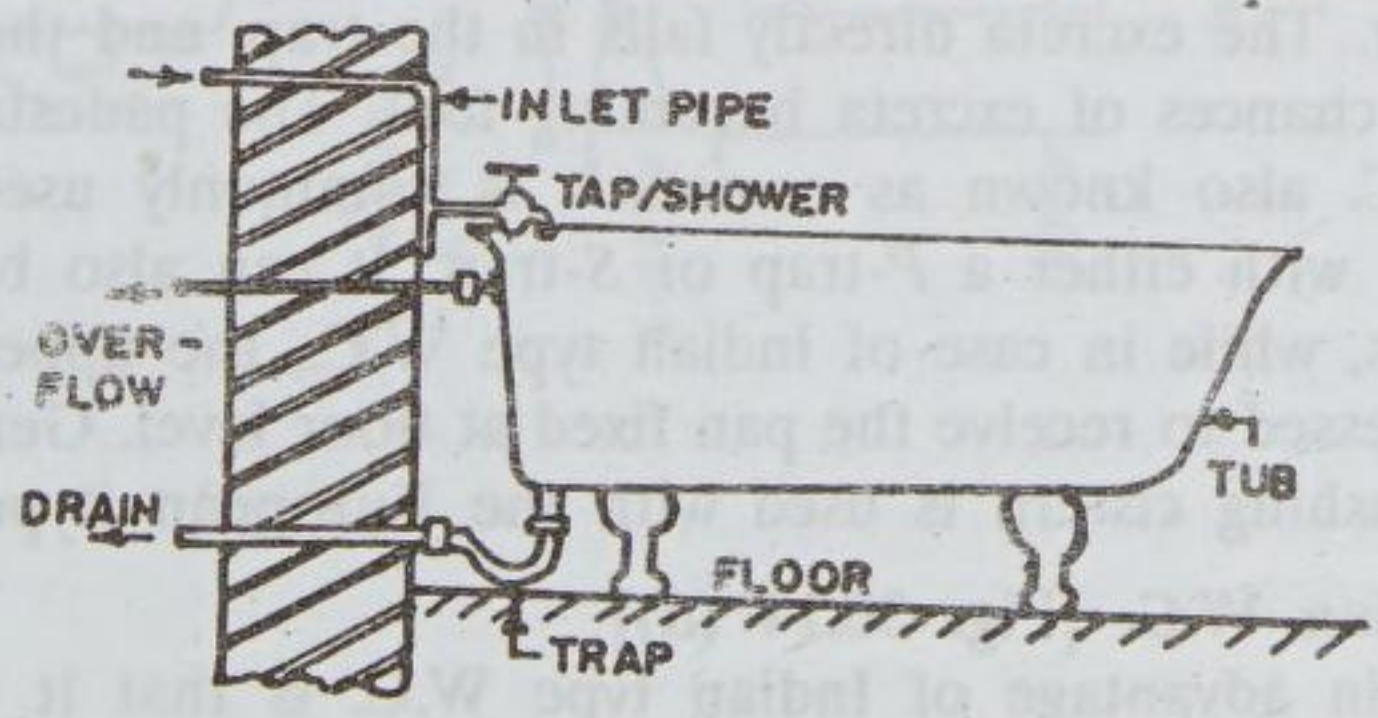


FIG. 30.13. BATH TUB.

else may be made of marble slabs properly jointed at the side. It has a length varying from 1.7 to 1.85 m, width between 0.7 m to 0.75 m and depth near waste pipe varying from 0.43 m to 0.45 m. The overall height may vary between 0.58 to 0.6 m. It is provided with outlet and overflow pipes, usually of 40 mm diameter. A trap with proper waterseal is used at the outlet.

4. Water closets

Water closets are designed to receive and discharge human excreta directly from the person using it. The appliance is connected to the soil pipe by means of a suitable trap. It is usually connected to a flushing cistern to flush the closet and discharge the human excreta to the soil pipe. Water closets are of three types.

(*i*) Indian-type

(*ii*) European type

(*iii*) Anglo-Indian type.

1. Indian type W.C. (Fig. 30.14 *a*)

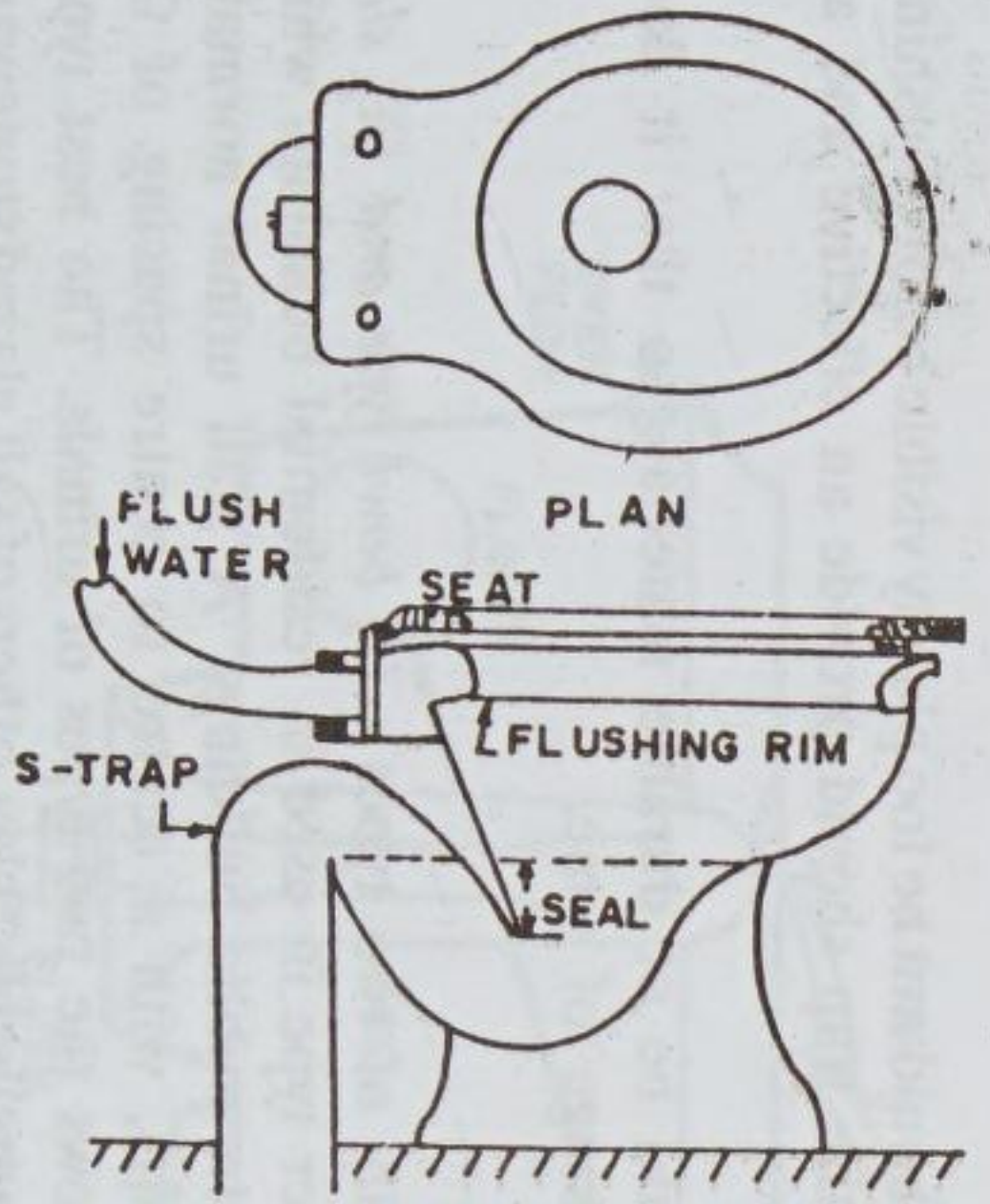
The Indian style water closet (W.C.), shown in Fig. 30.14 (a) is simple in construction and working, but is used in squatting position. It is usually made of porcelain. The pan and trap are available in two different pieces. The trap has an opening for antisiphonage pipe. The W.C. is fixed in squatting (or siting) position just at floor level. Since the excreta does not directly fall into the trap, therefore, there are chances for excreta to become foul. The excreta may stick to the surface of the pan if the flushing is not proper. The flush water enters the rim of the pan through the opening provided in the front of the pan. The flushing cistern is normally kept 2 m above the closet. Indian type closet requires greater quantity of water (atleast 10 litres) for flushing.

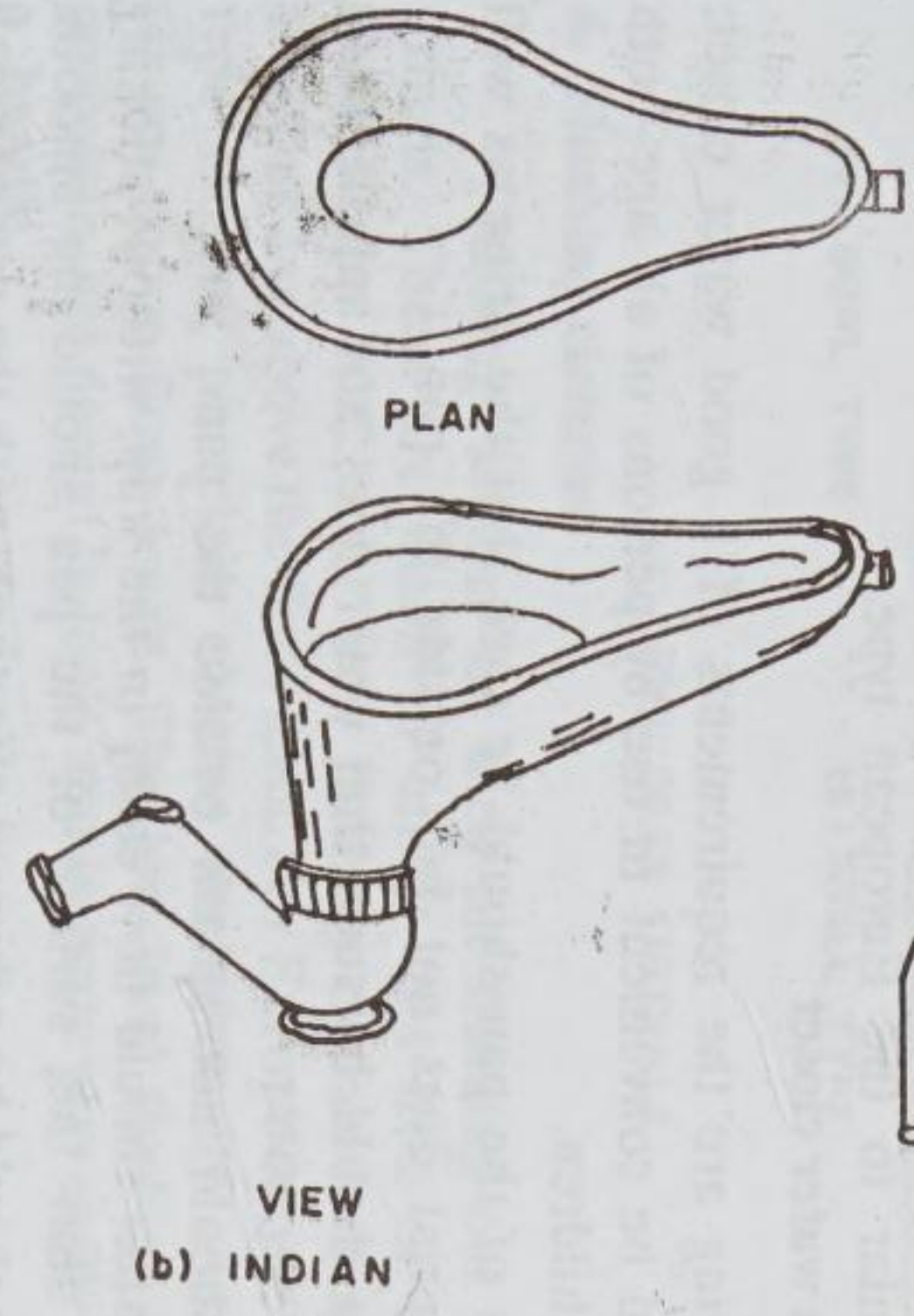
2. European type W.C.

Fig. 30.14 (*b*) shows a typical European type water closet. It is usually made of porcelain. It is a wash down water closet, provide with a seat and a cover. The pan has flushing rim to spread the flush water. The excreta directly falls in the trap, and therefore there are less chances of excreta becoming foul. The padestal type European W.C. also known as *commode* is commonly used. The closet is fitted with either a *P*-trap or *S*-trap. It can also be used at upper floors, while in case of Indian type W.C., the upper floor has to be depressed to receive the pan fixed at floor level. Generally, a low level flushing cistern is used with the European ltype W.C.

3. Anglo-Indian W.C. [Fig. 30.14 (*a*)]

The main advantage of Indian type W.C. is that it can be used in squatting position since it is fixed at floor level, while an





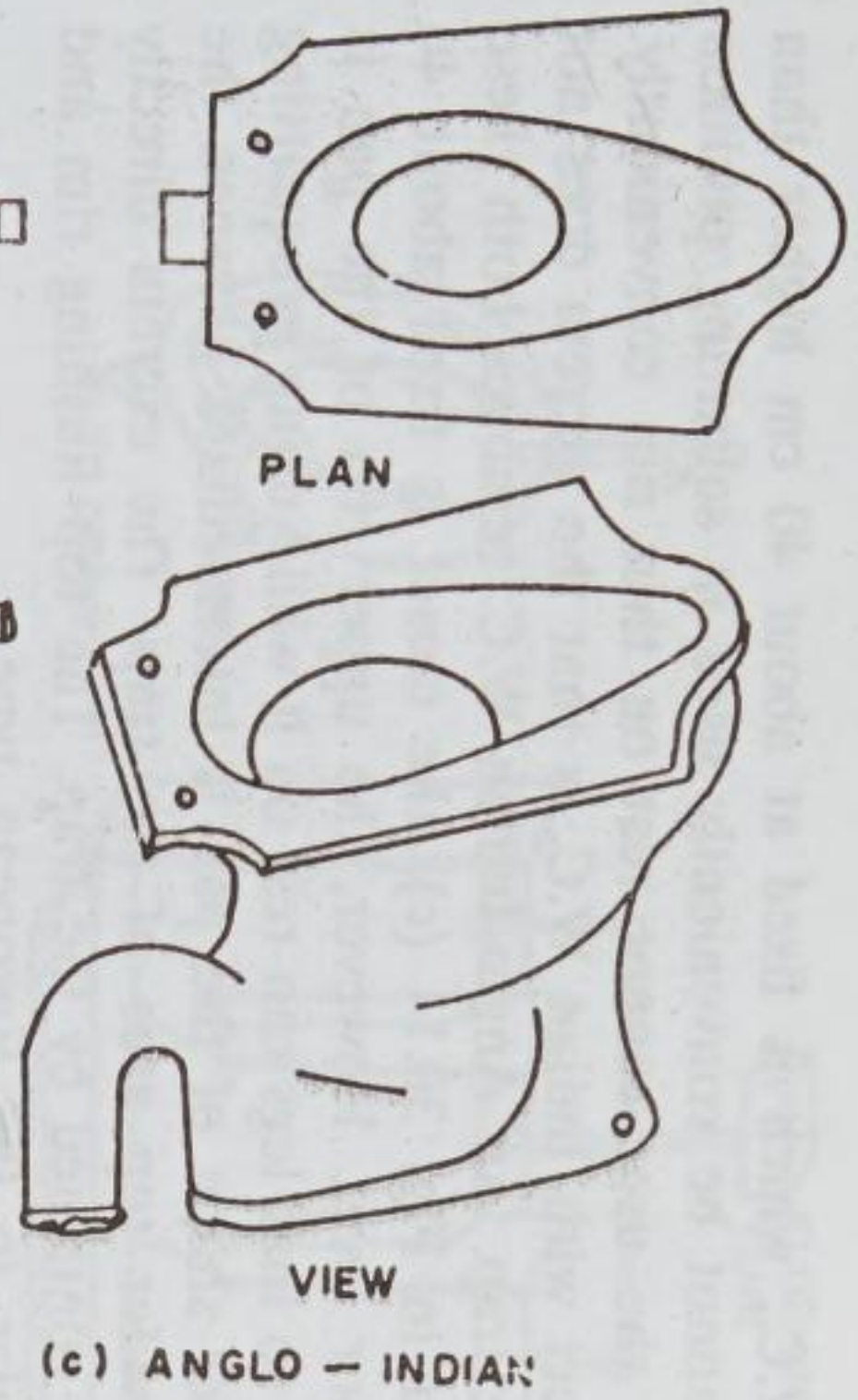


FIG. 30.14. WATER CLOSETS.

European type W.C., which is fixed at about 40 cm higher than the floor level, cannot be conveniently used in squatting position since the legs of the user cannot rest on thin rim conveniently. However, the defect with Indian W.C. is that the excreta does not fall directly in the trap. An Anglo-Indian W.C. removes both these defects. As shown in Fig. 30.14 (*c*), the closet is fixed about 40 cm above the floor level. However, the upper rim of the pan is properly enlarged so that legs can rest on it while using in squatting position. The inner shape of the pan is intermediate between the two types, with wider top area of the trap. The excreta directly falls in the water contained by the trap. The top flushing rim and seat etc. are similar to the European type.

Requirements of a water closet

The following are the requirements of a good water closet:

1. It should be convenient in use by persons of all age–both old as well as children.

2. The size of the pan should be such that the urine as well as the fecal material does not fall outside the pan.

3. The trap should be such that water does not splash when the excreta falls in water.

4. Urine should not splash outside the pan.

5. Fecal matter should flow easily in the trap, without sticking to the pan. For that the, surface of the pan should be smooth.

6. Flushing should be achieved effectively with the use of small quantity of water.

7. Fecal material should not be too plainly visible before flushing.

8. The water in the trap should provide an effective and air tight seal.

9. The pan should be of durable material, so that it does not crack with the passage of time.

5. Urinals

Urinals are usually of two types : (*i*) *bowl type and* (*ii*) *slab* or *stall type* . The former type in used in residential buildings while the later type is used in public buildings. A stall urinal normally has more than one units, with a centre to centre spacing of 0.6 to 0.7 m. Fig. 30.15 shows the two types of urinals. The best types of urinals are made of enamelled fireclay, others of salt glazed stoneware, marble, slate and in cement. The contents of urinals are collected and discharged into the soil pipe through floor trap (nahni trap). Automatic flushing cisterns are generally provided for stall type urinals, which operates, at regular interval of 10 to 15 minutes.

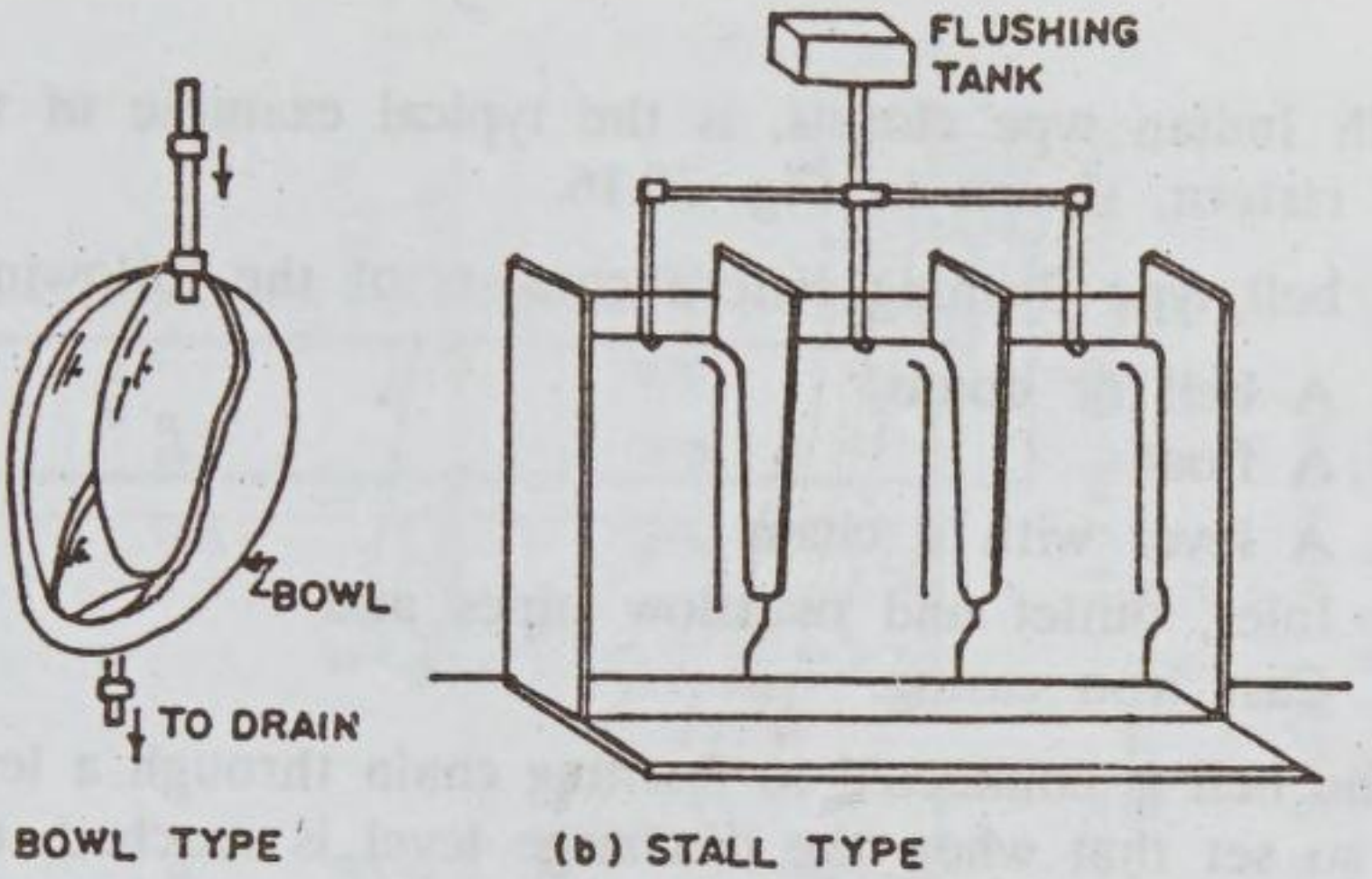


FIG. 30.15. URINALS.

6. Flushing cisterns

Flushing cisterns are used for flushing out water closets and urinals. These are made of either cast iron or of porcelain. For Indian type W.C., cast iron flushing cistern is normally used, fixed at about 2 m above the floor level. For European type and Anglo-Indian type closets, porcelain cisterns are normally used, fixed at about 60 cm above floor levels. The low level flushing cisterns, made of porcelain, are decent in look, and operate very easily by simply turning a handle.

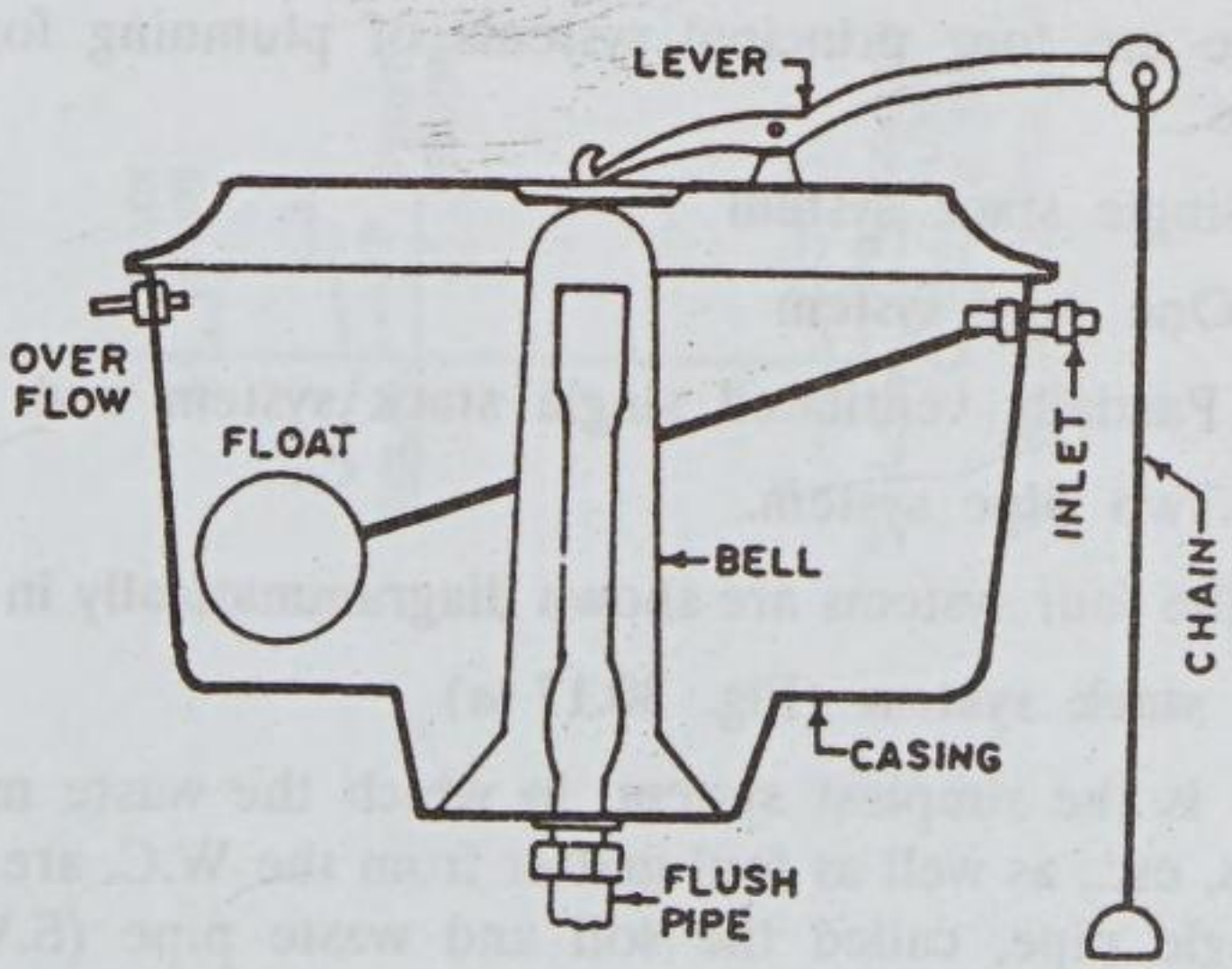


FIG. 30.16. BELL TYPE FLUSHING CISTERN

Flushing cisterns are of two types : (*i*) *valveless* siphonic type and (*ii*) *valve fitted* siphonic type. *Bell type* flushing cistern, commonly

used with Indian type closets, is the typical example of valveless siphonic cistern, shown in Fig. 30.16.

A bell type flushing cistern consists of the following parts:

1. A bell or dome
2. A float
3. A lever with a chain
4. Inlet, outlet and overflow pipes and
5. Cast iron casing.

The bell is connected to flushing chain through a lever. The float is so set that when the discharge level is reached, the float rod slightly closes the inlet cock. When the chain is pulled, the bell is lifted up, thus splashing the water. The splashing of water takes away some air with it, causing partial vacuum in the top of the bell. Siphonic action thus starts, and water in the cistern enters the bell through holes provided at its bottom. When the tank is emptied, air enters from the bottom and siphonic action is broken. The lowering of the float results in the opening of the inlet cock, and water thus enters the cistern. It should be noted that the chain should be released immediately after the pull, otherwise the partial vacuum caused by splashing water may be destroyed by the entry of air from the flush pipe. The capacity of a bell type flushing cistern may vary between 5 to 15 litres.

30.12. SYSTEMS OF PLUMBING

There are four principal systems of plumbing for drainage of buildings :

(*i*) Single stack system

(*ii*) One pipe system

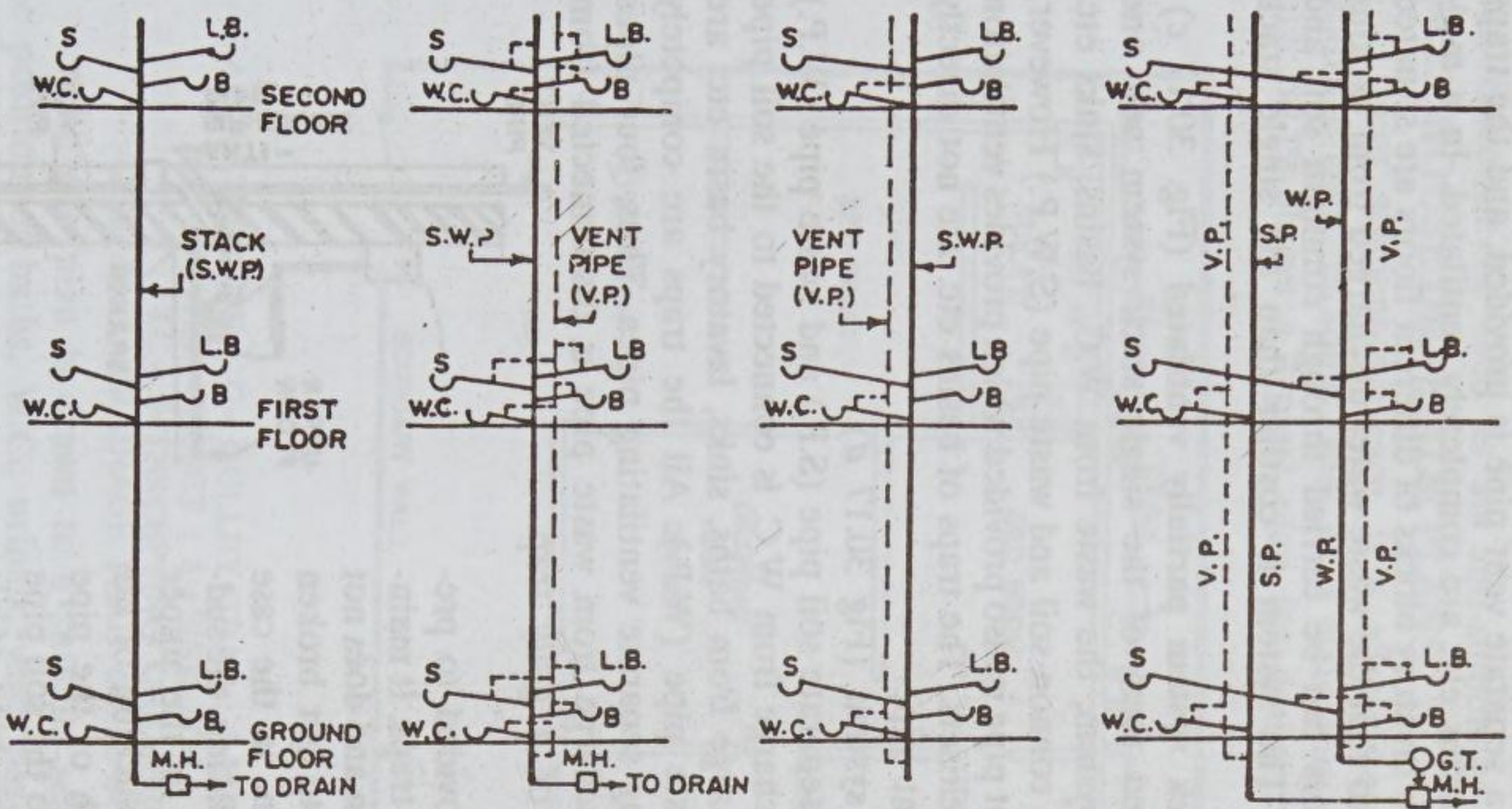
(*iii*) Partially ventilated single stack system and

(*iv*) Two pipe system.

All the four systems are shown diagrammatically in Fig. 30.15.

1. Single stack system (Fig. 30.17 *a*)

This is the simplest system, in which the waste matter from baths, sinks, etc., as well as foul matter from the W.C. are discharged in one single pipe, called the soil and waste pipe (S.W.P.). This pipe terminates as the vent pipe at its top, and no separate vent pipe is provided. The single stack system is effective only if the traps are filled with water seal of depth not less than 75 mm. Gulley traps and waste pipes are completely dispensed with. The system simple and economical since only one pipe is used.



(a) SINGLE STACK SYSTEM (b) ONE PIPE SYSTEM (c) SINGLE STACK PARTIALLY VENTILATED. (d) TWO PIPE SYSTEM

W.C.= WATER CLOSET ; B= BASIN ; L.B.= LAVATORY BASIN ; S= SINK ;
V.P= VENTILATING PIPE ; S.W.P.= SOIL PIPE AND WASTE PIPE ; S.P. = SOIL PIPE
W.P. =WASTE PIPE ; M.H. = MAN HOLE ; G.T.= GULLEY TRAP.
NOTE. ALL PIPES TERMINATE AS VENT PIPES AT THE TOP.
FIG. 30.17. PLUMBING SYSTEMS.

2. One pipe system (Fig. 30.17 *b*)

In this system, a separate vent pipe is provided, and the traps of all water closets, basins etc. are completely ventilated. In a multistoreyed building, the lavatory blocks of different floors are situated one above the other, so that the waste water discharged from various units at different floors can be carried through common soil and waste pipe (S.W.P.). The system is costlier than the single stack system.

3. Single stack system partially ventilated (Fig. 30.17 *c*)

This is modified form of the single stack system and one pipe system. In this system, the waste from W.C., basins, sinks etc. is discharged into one connon soil and waste pipe (S.W.P.) However, in addition, a relief vent pipe is also provided which provides ventilation to the traps of water closets. The traps of basins etc. are not directly connected to the vent pipe.

4. Two pipe system (Fig 30.17 *d*)

In this system, separate soil pipe (S.P.) and waste pipe (W.P.) are provided. The discharge from W.C. is connected to the soil pipe (S.P.) while the discharge from baths, sinks, lavatory basin etc. are connected to the waste pipe (W.P.). All the traps are competely ventilated by providing separate ventilating pipes. Thus, four pipes are required. The discharge from waste pipe is disconnected from the drain by means of a gully trap.

Anti-siphonage pipe

It is a pipe provided to preserve the water seal of traps. It maintains proper ventilation and does not allow the water seal to get broken due to siphonic action. In the case of a multi- storeyed building, the sudden flush of water in the upper storey results in the sucking of air from the short branch of the pipe connecting the W.C. to the soil pipe of lower storey. This sucking of air causes partial vacuum on the downstream side of the water seal of the lower W.C. The pressure at the upstream side of the water seal is more (atmospheric), which forces the water up the trap and siphons it out in the branch. This results in breaking of the water seal. This can be avoided by connecting the crown of the trap to the atmosphere

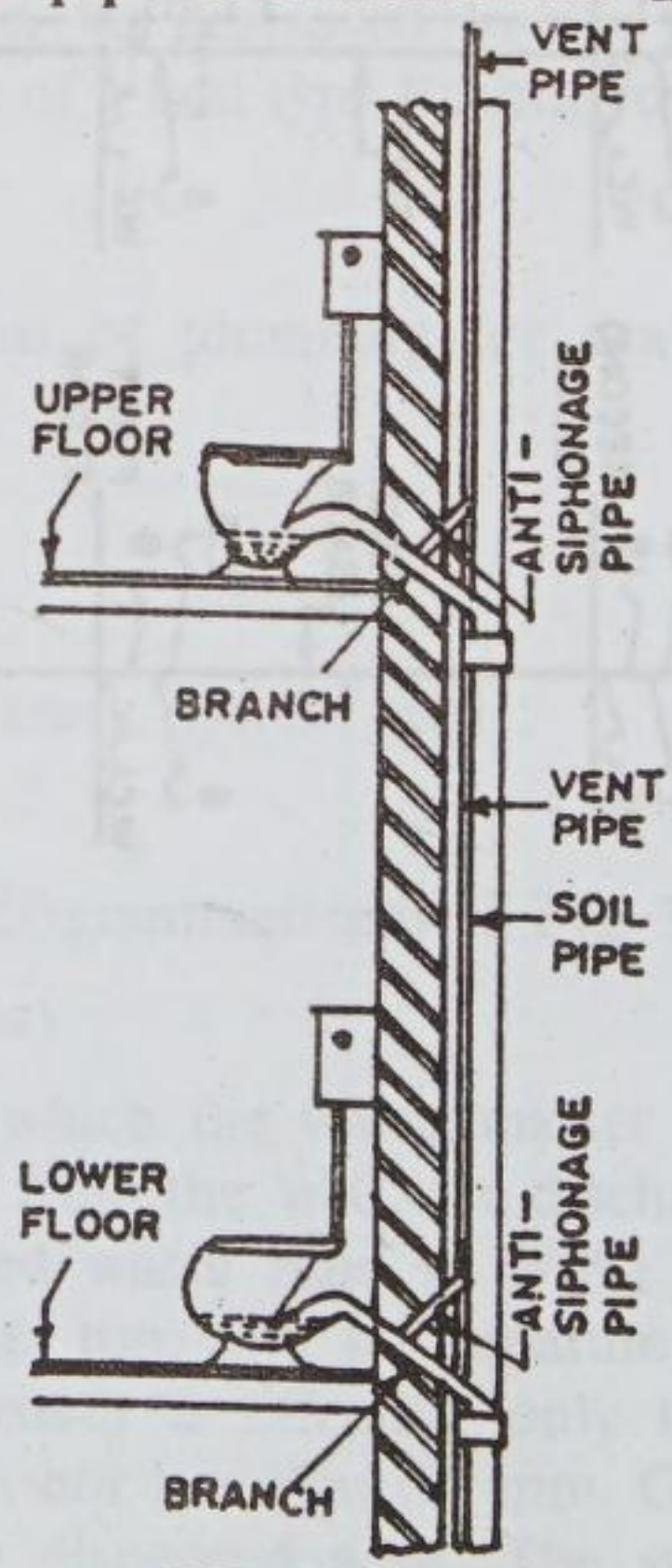
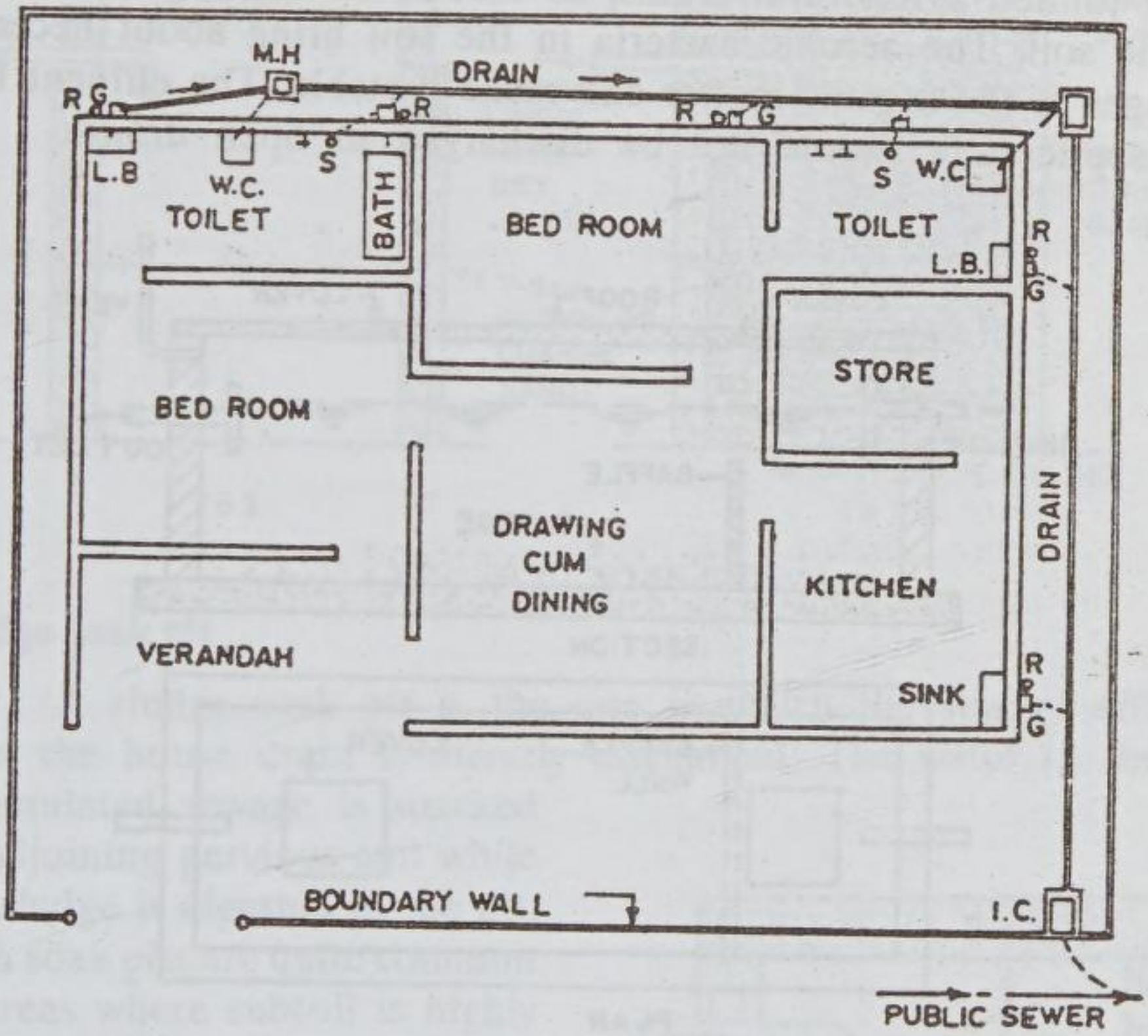


FIG. 30.18.
ANTI-SIPHONAGE PIPE.

through an anti-siphonage pipe (Fig. 30.18). A ventilating pipe can therefore be used as an anti-siphonage pipe.

30.13. HOUSE DRAINAGE PLANS

For efficient drainage, it is always better to prepare house drainage plan. In some cities, it is statuatory to submit such plans. Fig. 30.19 shows a typical plan for drainage of a small house.



R= RAINWATER PIPE ; G= GULLEY TRAP ; L.B.= LAVATORY BASIN
W.C.= WATER CLOSET ; S= SINK M.H.= MANHOLE;
I.C.= INTERCEPTING CHAMBER.

FIG. 30.19. DRAINAGE PLAN OF A BUILDING.

The site plan is drawn to a suitable scale, showing onto it the position of baths, W.C., urinals, wash basins and other units, along with the position of gully traps and floor traps. The longitudinal section of the drain is also drawn, showing distances, invert levels, size and levels of inspection chambers and man holes, gradient of pipes and the position and level of the public sewer.

30.14. SEPTIC TANK

A septic tank is used to treat sewage from isolated group of country houses, where a piped sewage system (*i.e.*, a public sewer) is not available. It is a horizontal continuous flow sedimentation tank in which sewage moves very slowly. Septic tank serves two purposes : (*i*) deposition of the settling solids in sewage by sedimen-

tation, and (*ii*) partial or complete digestion of the sludge prior to its disposal. A septic tank produces septic action by anaerobic bacteria, wherein proteins, carbohydrates, cellulose and fatty matter present in sewage are broken to simpler compounds. The nitrogen is converted to ammonia, while the colloidal matter is flocculated, then liquified and finally digested. The effluent from the septic tank is discharged either into soak pit or below ground level through open jointed agricultural drains, so that the effluent gets absorbed in the soil. The aerobic bacteria in the soil bring about necessary changes in the organic matter and make it stable. The effluent from the septic tank should not be discharged in open drains.

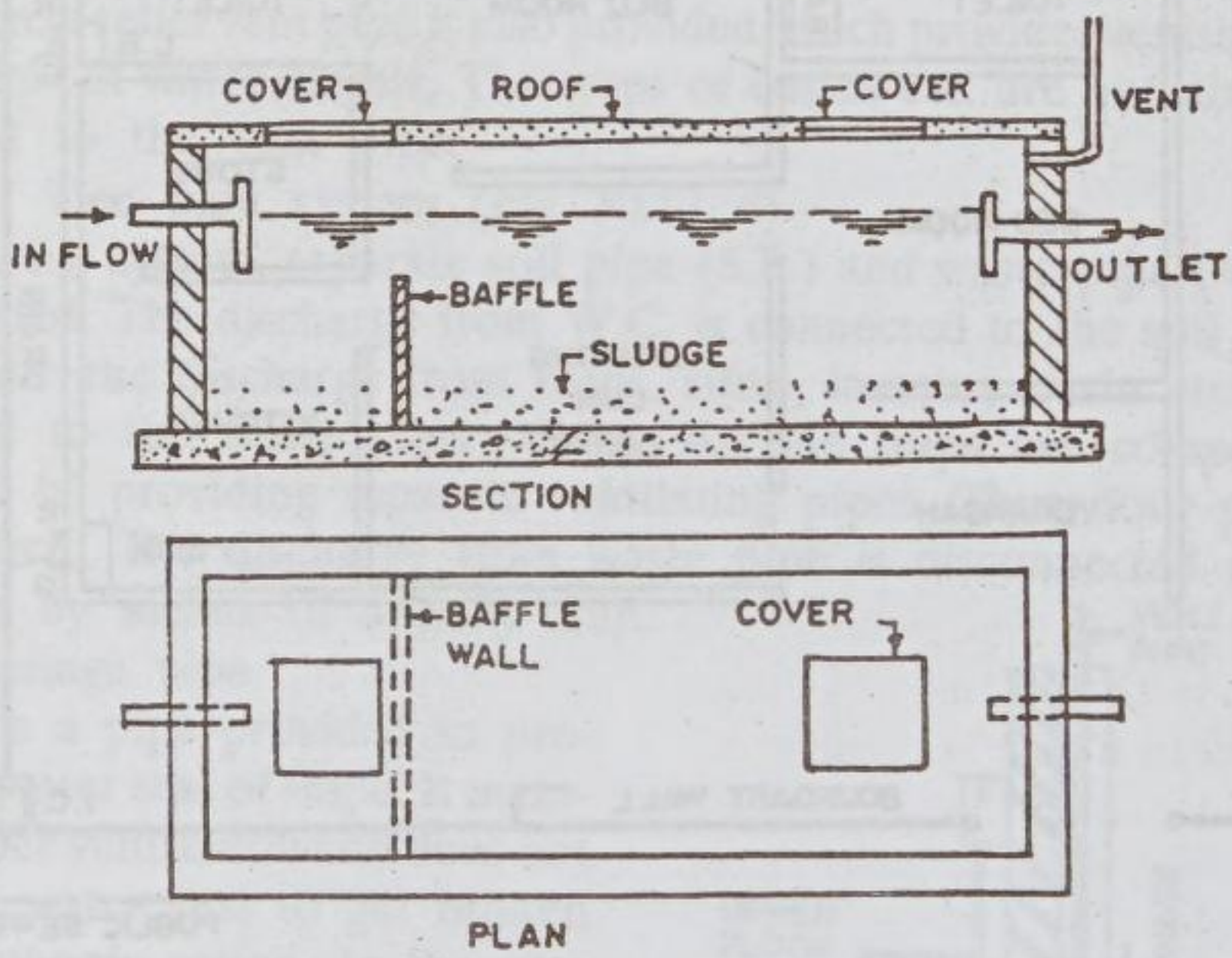


FIG. 30.20. SEPTIC TANK.

Fig. 30.20 shows a simple septic tank. It is so constructed that direct current between inlet and outlet is prevented. This is achieved by using pipe-tees at inlet and outlet. A baffle wall with openings is constructed at some distance away from the outlet. The outlet pipe is kept about 15 cm lower than the inlet pipe. The sludge, which is deposited in the bottom is cleaned periodically, say once in 6 to 12 months. A vent pipe is provided for the escape of gases.

30.15. SOAK PIT (SEEPAGE PIT)

A soak pit is a covered pit dug in the ground, in which effluent from the septic tank is discharged. It is generally dug in the pervious soil which can absorb the effluent. Fig. 30.21 shows two forms of soak pits : (*a*) lined, and (*b*) unlined.

Lined soak pits are used when the inner volume is kept empty.

The lining may be of bricks, stone or concrete blocks, with dry open joints, and with atleast 7.5 cm backing of coarse aggregate. However, if the inner volume is filled with stone and brick aggregates, no lining is required, except for the masonry ring provided at the top.

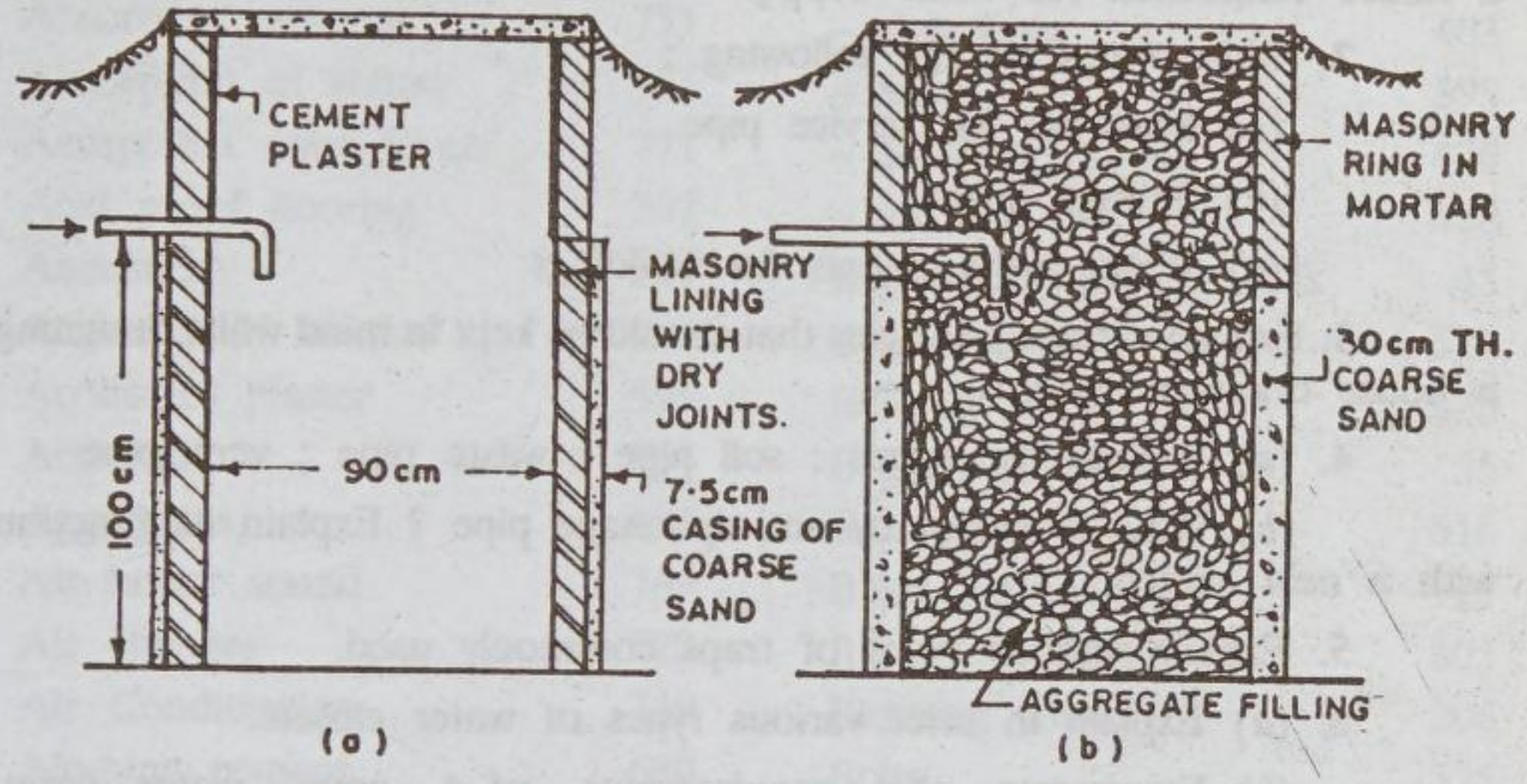


FIG. 30.21. SOAK-PITS.

Sludge soak pit

A sludge soak pit is the one in which the sewage effluent from the house drain is directly discharged. The water in the accummulated sewage is soacked by adjoining pervious soil while the sludge is digested in the pit. Such soak pits are quite common in areas where subsoil is highly pervious. Fig. 30.22 shows a typical section of such a soak pit. It is lined with masonry with open joints. The top is covered with a rigid slab. The diameter of such a tank varies from 2 m to 3 m, while the depth varies from 2.5 m to 4 m. Each individual house has one such soak pit which has a life of about 20 years for a family of 6 persons. When the soakpit gets filled up, the top cover is removed and the tank is emptied.

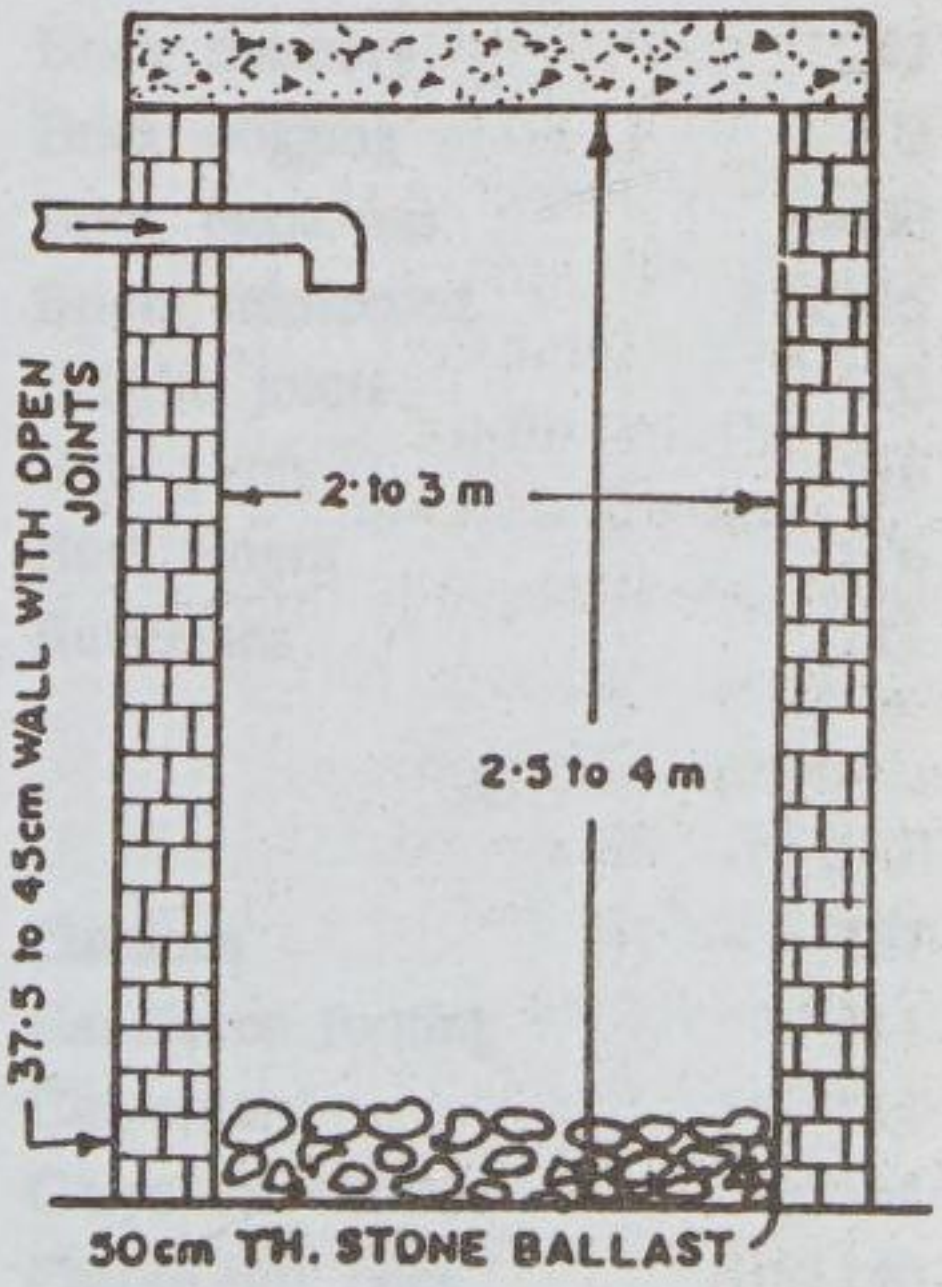


FIG. 30.22. SLUDGE SOAK PIT.

PROBLEMS

1. Describe, with the help of a neat sketch, the method of taking a house connection for water supply.

2. Write notes on the following :

(*a*) Materials for service pipe

(*b*) Storage tanks

(*c*) Water requirements for buildings

3. Explain various principles that should be kept in mind while designing a house drainage system.

4. (*a*) Explain the terms : soil pipe ; waste pipe ; vent pipe

(*b*) What is meant by anti-siphonage pipe ? Explain its function with a neat sketch.

5. Explain various types of traps commonly used.

6. (*a*) Explain in brief various types of water closets.

(*b*) Enumerate the requirements of a good water closet.

7. Explain, with the help of a diagram, the working of a Flushing cistern.

8. Explain, with the help of diagrams, various systems of plumbing used for house drainage.

9. Draw a typical house drainage plan of a residential building.

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